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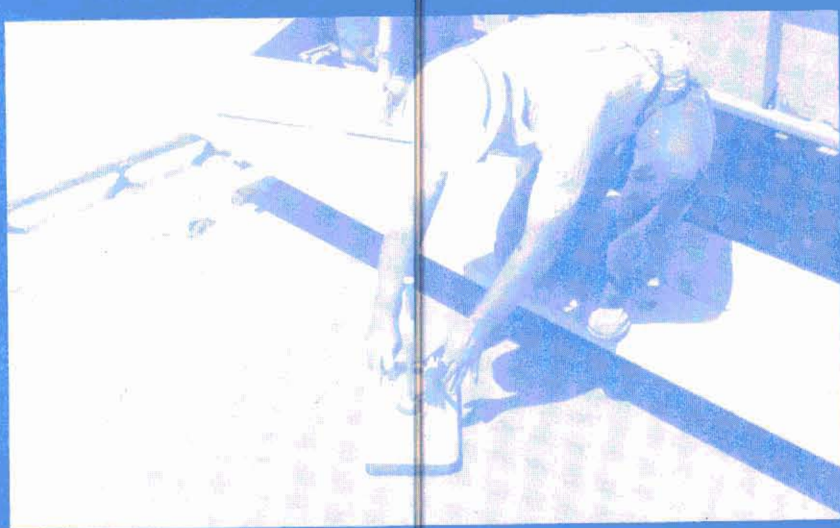
Relationship of Consolidation to Performance of Concrete Pavements



U.S. Department of Transportation
Federal Highway Administration

Research, Development, and Technology
Turner-Fairbank Highway Research Center
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FOREWORD

This report summarizes research on the relationship of degree of consolidation to critical performance properties of portland cement concrete (PCC) pavements and on the suitability of nuclear density gauges for monitoring consolidation of such pavements. It will be of interest to materials and construction engineers concerned with PCC construction.

The Federal Highway Administration (FHWA) appreciates the assistance of the Idaho, Iowa, and Illinois Departments of Transportation during observations of nuclear density gauge testing.

Copies of this report are being distributed by FHWA transmittal memorandum. Sufficient copies are being distributed to provide a minimum of one copy to each regional and division office and two copies to each State highway agency. Direct distribution is being made to the division offices. Additional copies may be obtained from the National Technical Information Service, 5285 Port Royal Road, Springfield, Virginia 22161.

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Federal Highway Administration

NOTICE

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16. Abstract

A study was made of the influence of consolidation on properties of portland cement concrete. Consolidation was found to have a strong influence on compressive strength, bond of concrete to reinforcing steel, and permeability of concrete. There is a lesser effect of consolidation on resistance to freezing and thawing. There is a loss of about 30 percent in compressive strength for every 5-percent decrease in consolidation.

A variety of nuclear density gauges were evaluated for use in monitoring consolidation of concrete. Use of these gauges has remained fairly constant, at relatively low levels, since 1977. A combination of techniques, such as consolidation monitoring device (CMD) and commercial direct transmission gauges, shows promise as a means of monitoring consolidation during the paving process.

A model acceptance sampling plan for concrete consolidation is proposed. The plan is of the inspection by variables type and requires a sample size of eight per lot. The plan provides for buyer's and seller's risks of 5 percent. The plan was field tested in Idaho and Iowa. Field testing indicated that monitoring concrete pavement consolidation is practical and economically feasible.

17. Key Words Acceptance tests, Bond, Compressive strength, Concretes, Consolidation, Freeze-thaw durability, Nuclear density gauges, Permeability, Quality control, Specifications.	18. Distribution Statement No restrictions. This document is available to the public through the National Technical Information Service, Springfield, Virginia 22161.

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METRIC (SI*) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

LENGTH

Symbol	When You Know	Multiply By	To Find	Symbol
in	inches	2.54	millimetres	mm
ft	feet	0.3048	metres	m
yd	yards	0.914	metres	m
mi	miles	1.61	kilometres	km

AREA

Symbol	When You Know	Multiply By	To Find	Symbol
in^2	square inches	645.2	millimetres squared	mm^2
ft^2	square feet	0.0929	metres squared	m^2
yd^2	square yards	0.836	metres squared	m^2
mi^2	square miles	2.59	kilometres squared	km^2
ac	acres	0.395	hectares	ha

MASS (weight)

Symbol	When You Know	Multiply By	To Find	Symbol
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams	Mg

VOLUME

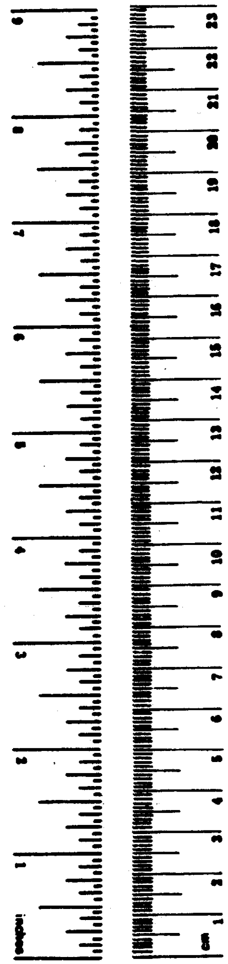
Symbol	When You Know	Multiply By	To Find	Symbol
fl oz	fluid ounces	29.57	millilitres	mL
gal	gallons	3.785	litres	L
ft^3	cubic feet	0.0328	metres cubed	m^3
yd^3	cubic yards	0.0765	metres cubed	m^3

NOTE: Volumes greater than 1000 L shall be shown in m^3.

TEMPERATURE (exact)

Symbol	When You Know	Multiply By	To Find	Symbol
°F	Fahrenheit temperature	5/9 (after subtracting 32)	Celsius temperature	°C

* SI is the symbol for the International System of Measurements



APPROXIMATE CONVERSIONS TO SI UNITS

LENGTH

Symbol	When You Know	Multiply By	To Find	Symbol
mm	millimetres	0.039	inches	in
m	metres	3.28	feet	ft
m	metres	1.09	yards	yd
km	kilometres	0.621	miles	mi

AREA

Symbol	When You Know	Multiply By	To Find	Symbol
mm^2	millimetres squared	0.0016	square inches	in^2
m^2	metres squared	10.764	square feet	ft^2
km^2	kilometres squared	0.39	square miles	mi^2
ha	hectares (10 000 m^2)	2.53	acres	ac

MASS (weight)

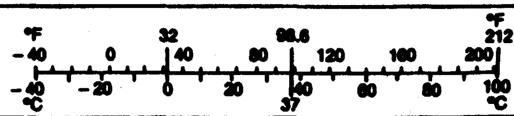
Symbol	When You Know	Multiply By	To Find	Symbol
g	grams	0.0353	ounces	oz
kg	kilograms	2.205	pounds	lb
Mg	megagrams (1 000 kg)	1.103	short tons	T

VOLUME

Symbol	When You Know	Multiply By	To Find	Symbol
mL	millilitres	0.034	fluid ounces	fl oz
L	litres	0.264	gallons	gal
m^3	metres cubed	35.315	cubic feet	ft^3
m^3	metres cubed	1.308	cubic yards	yd^3

TEMPERATURE (exact)

Symbol	When You Know	Multiply By	To Find	Symbol
°C	Celsius temperature	9/5 (then add 32)	Fahrenheit temperature	°F



These factors conform to the requirement of FHWA Order 5190.1A.

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INTRODUCTION

Portland cement concrete (PCC) has proven to be a versatile, strong, and durable material for construction of highway pavements. The combination of traffic loading, wear, temperature stresses, freezing, and applications of deicing agents, however, results in a severe environment which requires the use of high quality concrete materials and practices if the pavement is to perform as designed over its service life. Inadequate consolidation is but one very important factor which can lead to premature concrete deterioration requiring expensive rehabilitation or replacement. A knowledge of the effects of incomplete consolidation on various concrete properties is needed so that the relationship between such factors as strength and degree of consolidation can be quantified and used by State highway agencies in preparation of quality control specifications.

LITERATURE REVIEW

Prior to initiation of laboratory studies, a literature study was conducted. This study was designed to obtain information on the effect of consolidation on properties of concrete important in highway applications. The properties of interest included strength (compressive and flexural), bond to reinforcing steel, permeability, and freeze-thaw durability.

1. General

[redacted] Fresh concrete, when initially placed, is usually honeycombed with entrapped air. Consolidation, usually achieved through mechanical vibration, is needed to eliminate these voids which otherwise would result in a weak, porous, and nondurable material. There are many excellent publications which describe the technique and equipment needed to properly consolidate concrete. These include: an NCHRP synthesis, the ACI Standard Practice for Consolidation of Concrete, and other review articles.[1,2,3,4] There is little detailed, quantitative information in these sources, however, describing how important concrete properties vary with degree of consolidation. For this reason, original literature sources were consulted in the various areas of interest. Information obtained is described below.

2. Effect of Consolidation on Strength of Concrete

Perhaps the earliest (and most widely utilized study) is that of Glanville.[5] While this study had as its main objective the determination of effect of aggregate gradation on strength, some limited work was done with partial compaction of concrete. Glanville's relationship is reproduced as figure 1. These data indicate a drop of approximately 30 percent in strength (measured on 6-inch (150 mm) cube specimens) for a decrease in degree of consolidation from 100 to 95 percent. A decrease of 10 percent in degree of consolidation is accompanied by a decrease of almost 60 percent in strength. Results obtained by Kaplan in a later study were virtually identical, showing a 30 percent drop in compressive strength for a decrease in degree of consolidation of 5 percent and a 50 percent strength decrease for a 10 percent reduction in consolidation.[6]

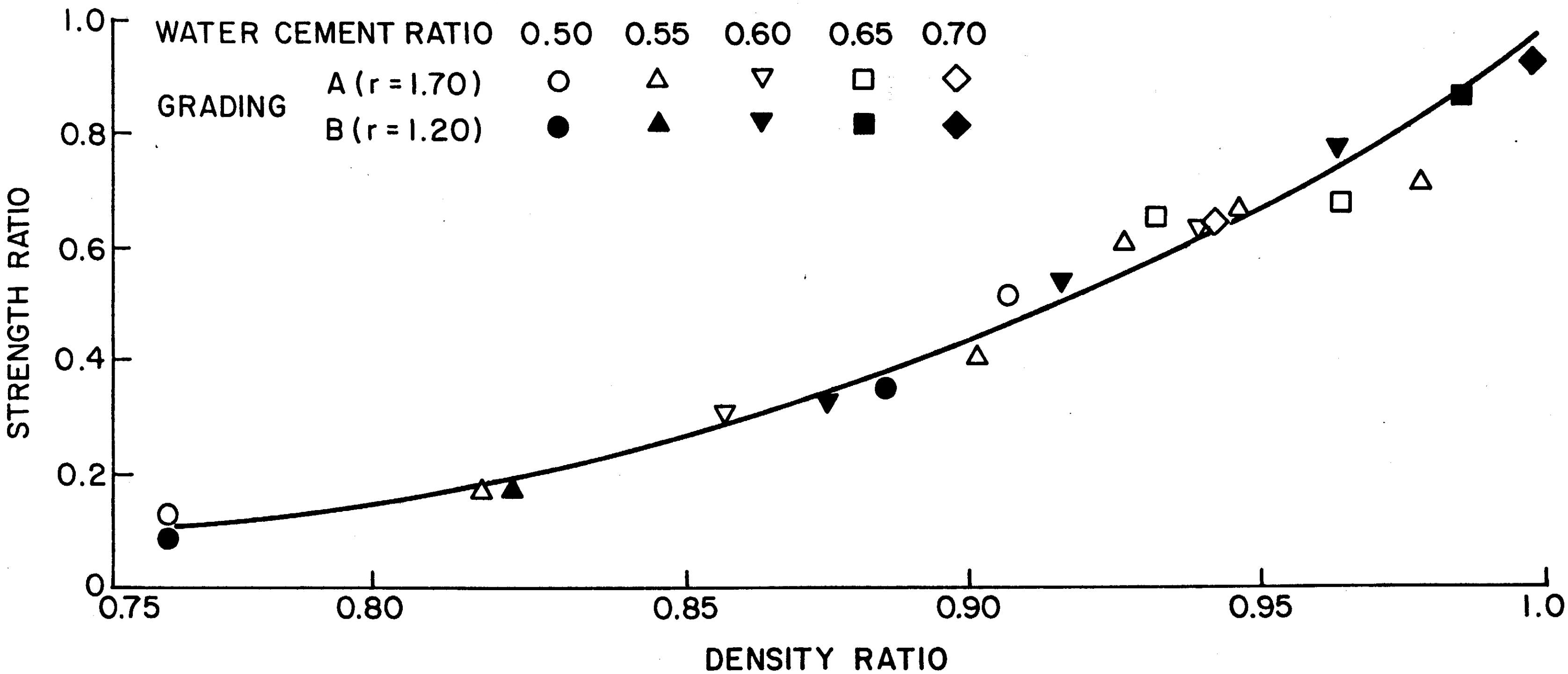


Figure 1. Relation between strength ratio and density ratio (reference 5).

Kaplan also reported a flexural strength reduction of 24 percent at 5 percent decrease in consolidation and 45 percent for a 10 percent decrease, these values being somewhat less than corresponding reductions in compressive strength. Other studies in this area include those by Stewart[7] and Talbot,[8] where similar results were obtained.

In addition to laboratory studies, a limited amount of field data relating consolidation to concrete strength are also available. McBride reported a loss of approximately 1,000 psi (6.9 MPa) in compressive strength of pavement cores for a reduction of 5 lb/ft^3 (80 kg/m^3) in density.[9] Gerhardt noted an average difference of compressive strength of approximately 400 psi (2.8 MPa) when cores were obtained from concrete directly in the path of paver mounted vibrators as opposed to cores obtained from areas between the vibrators.[10]

3. Effect of Consolidation on Bond to Reinforcing Steel

There is very little quantitative information relating to the effects of consolidation on bond of concrete to reinforcement. While studies on "revibration" indicate that a reduction in void space between reinforcement and concrete (which is a major benefit of revibration) is indeed beneficial from the point of bond, this additional consolidation is normally applied a number of hours after the concrete is placed; therefore, the effects may not be the same as when degrees of consolidation of concrete vibrated immediately after placement are compared.[11]

An indication of the effects of consolidation on bond may be inferred from the work of Hognestad and Siess.[12] In these studies, pullout tests were performed on deformed bars in various configurations. While the major variable was the level of entrained air, as a first approximation one may assume that an increase in entrained air content will have an effect on bond similar to that of a decrease in degree of consolidation. For these studies, an increase of 1 percent in air content resulted in a decrease of 10 percent in bond.

4. Effect of Consolidation on Freeze-Thaw Durability of Concrete

Most studies dealing with effects of consolidation on freeze-thaw durability have been concerned with the effects of overconsolidation, that is, how prolonged vibration influences durability. The consensus with regards to this issue, as demonstrated in studies by Tynes[13] and Backstrom,[14] is that, while vibration may remove some of the entrained air from concrete, it is mostly the larger sizes of voids which are displaced from the concrete, and the remaining void system, while having lower total air content, generally exhibits spacing factors within acceptable limits. [redacted]

In spite of an extensive literature search, no documented information concerning the effects of incomplete consolidation on freeze-thaw durability was found.

5. Effect of Consolidation on Structural Response of Concrete Pavements

Inadequate consolidation of concrete may affect concrete pavement structural response and, therefore, performance as follows:

- Reduced service life due to lower strength.
- Poor performance at doweled joints.
- Effect on cracking in continuously reinforced concrete pavements.

a. Effect on Service Life

[redacted]. For equal design traffic, a pavement with lower strength concrete would require a larger pavement thickness. Conversely, if, because of poor consolidation, the as-built concrete strength is lower than required by design, then the pavement service life can be expected to be significantly lower.

An illustration of the effect of concrete strength on pavement service life is given below using the procedures detailed in the "AASHTO Guide for Design of Pavement Structures."[15]

The following design parameters were used to project service life of pavement sections with different concrete strengths:

- Design Slab Thickness = 10 inches (250 mm)
- Modulus of Subgrade Reaction = 300 pci (0.08 N/mm^3)
- Drainage Coefficient, C_d = 1.0
- Load Transfer Coefficient, J = 3.2 (Doweled)
- Concrete Modulus of Elasticity = 4,200,000 psi (29 GPa)
- Terminal Pavement Serviceability Index = 2.5

- Design Reliability, R = 90 percent
- Overall Standard Deviation (for design equation) = 0.30.

Using this approach the projected service life of pavement sections could range from 20 years for a concrete flexural strength of 600 psi (4.1 MPa) to approximately 6 years for a concrete flexural strength of 400 psi (2.7 MPa). This demonstrates that concrete strength has a significant effect on anticipated service life of a pavement. [redacted].

Loss in serviceability generally occurs due to development of a larger amount of slab cracking due to reduced concrete strength. Slab cracking models for plain concrete pavement are generally of the following form:

CRACKING = f (ESAL, SR)

where: CRACKING = total length of cracks, ft/lane mile (m/lane Km)
ESAL = accumulated 18-kip (124 MPa) equivalent single-axle loads.
SR = ratio of edge stress/modulus of rupture computed for a reference wheel load

Based on a recent nationwide survey of plain concrete pavements,[16] concrete flexural strength was found to have a significant influence on the amount of midslab cracking.

Another way to consider the effect of lower strength is to consider the fatigue behaviour of plain concrete. For a given applied stress level, the stress ratio for a lower strength concrete is higher and this would result in a smaller number of load applications to failure. Several concrete pavement design procedures incorporate the fatigue behaviour of concrete for determination of required slab thickness.[17,18] If the effect of poor consolidation were to be accounted for in these procedures, then larger slab thicknesses would be required for a given set of design conditions.

b. Effect on Doweled Joints

Inadequate consolidation of concrete may result in poor performance of doweled joints. When dowel baskets are used, incomplete consolidation may result in honeycombed concrete under the dowels. [redacted]

[redacted] If concrete surrounding the dowels has voids or is not stiff enough, then the dowels may not be as effective in transferring load across a joint. This deficiency may ultimately result in faulting and pumping, as well as surface or bottom spalling, at joints.

While the effect of concrete strength on joint spalling is generally recognized, only limited data exist to directly relate doweled joint performance to concrete strength. Laboratory work by Marcus during the early 1950's indicated [redacted][19] This work indicated that the allowable bearing stress under a dowel should be about 80 percent to 100 percent of the concrete compressive strengths for dowels ranging in diameter from 1 to 1-1/2 inches (25 to 37 mm).

Thus, if concrete strength is lower than designed for, dowel failure in the form of spalling around the dowel may be expected.

In addition to bearing type failure, dowel looseness may develop earlier if concrete strength is low and this will result in poor load transfer across joints.

c. Effect on Continuously Reinforced Concrete Pavement (CRCP)

For continuously reinforced concrete pavements, the effect of inadequate consolidation affects two quantities, pavement serviceability and crack spacing.

The effect of low concrete strength on pavement serviceability is similar to that discussed previously (section 5-a).

For CRCP, concrete strength also affects crack spacing. [redacted] While smaller crack spacing results in cracks being tightly closed, there is a limit to how small the crack spacing can be in CRCP. [redacted]. The minimum crack spacing desired can be approximated by the following equation:[20]

$$L = \frac{0.5 \cdot d \cdot f_{sy}}{f_u} \tag{1}$$

where: L = minimum crack spacing, inch (mm)
d = reinforcing steel bar diameter, inch (mm)
f_{sy} = steel yield stress, psi (MPa)
f_u = concrete/steel bond stress, psi

As an example, for 3/4-inch (19 mm)-diameter Grade 60 (410 MPa) reinforcing bars, crack spacing would range from approximately 19 ft (5.8 m) for a bond stress
7 of 100 psi (0.7 MPa) to 4 ft (1.2 m) for a bond stress of 500 psi (3.5 MPa). Normally, crack spacings in CRCP range from 4 to 6 ft (1.2 to 1.8 m). A minimum bond strength of about 300 psi (2.1 MPa) is required for an average CRCP. It should be noted that lower concrete strength results in smaller crack spacing. Therefore, unless a minimum concrete strength is available, the CRCP may exhibit excessive distress due to concrete/steel bond failure.

d. Field Evaluation of the Effect of Consolidation on Pavement Structural Response

Concern with evaluation of the degree of consolidation of inplace concrete in pavements has generally been expressed in relation to construction of continuously reinforced concrete pavements (CRCP). It has been felt that because of the heavy reinforcing used and because of use of the two-lift placement technique, adequate consolidation of concrete may not be achieved on some CRCP projects. Many field studies of CRCP did confirm that inadequate consolidation was a problem on several early CRCP projects and that the problem resulted in honey-combed concrete below the reinforcing steel.(21,24) [redacted]

With experience gained over the years, problems due to poor consolidation are now less frequent. These problems have been minimized by most highway agencies by use of specifications for paving concrete that incorporate requirements for proper vibration of concrete.

However, there is still some risk that inadequate concrete consolidation may occur in plain, reinforced, or continuously reinforced concrete pavements because of higher placement rates, use of heavy reinforcement (CRCP), two-lift construction, inadequate specifications related to consolidation of concrete, and poor quality control at the site.

During a recent extensive study to relate quality control parameters with concrete pavement performance, an attempt was made to evaluate the effect of relative concrete density (consolidation) on pavement performance.[25] Quality control and performance data were collected from 104 projects from Florida, Louisiana, Maryland, New York, and Ohio. However, because concrete density data were not available for many projects, correlations were not developed between relative concrete density and pavement performance.

6. <u>Conclusions Drawn from Literature Survey</u>

Results of the literature survey indicate that consolidation may have a significant affect on material properties and field performance of concrete. [redacted]; however, the effects of voids on bond due to lack of consolidation have not been the subject of reported studies. [redacted]

Consolidation will affect pavement service life as reflected in its effect on flexural strength. Inadequate consolidation may lead to premature pavement failure. Consolidation can also affect load transfer across doweled joints, and poor consolidation can thereby decrease joint effectiveness.

More information is needed on the influence of mix variables such as aggregate type, cement content and air content on relationships between consolidation and important physical properties of concrete. To obtain such information a laboratory study was included as part of this research program.

LABORATORY STUDY: EFFECTS OF CONSOLIDATION ON SOME IMPORTANT PROPERTIES OF CONCRETE

1. Objectives and Scope

This research had two primary objectives. The first objective was to determine the relationships between degree of consolidation of concrete and its compressive strength, bond to reinforcing steel, chloride permeability, and resistance to freezing and thawing. The second objective was to determine the influence of such factors as cement content, air content, and aggregate type on these relationships, and to develop quantitative expressions which could be used in development of statistically based quality assurance specifications.

The objectives were carried out within the following scope:

1. Air-entrained concretes were prepared using gravel and crushed limestone aggregates over a range of cement contents encompassing those currently used in highway pavement construction.
2. Concretes were consolidated using standard practice (100 percent consolidation) and at nominal levels of 92 and 85 percent. In addition, a series of concretes were deliberately overconsolidated.
3. Concrete specimens were prepared and tested for 28-day compressive strength, 28-day bond to reinforcing steel (bond stress at 0.01-inch (0.25 mm) loaded-end slip), chloride permeability (AASHTO T 277: Rapid Method), and freeze-thaw resistance (ASTM C 666-Procedure A).
4. Data were statistically analyzed in order to develop quantitative relationships between degree of consolidation and the properties investigated. Expressions were developed to allow prediction of properties based on degree of consolidation, air content, and cement content of the mixtures.

2. Experimental

a. Materials

The cement used was a blend of three ASTM Type I cements available in the Chicago, Illinois, area. Physical and chemical properties of the cement blend are given in table 1.

Aggregates used were a predominantly dolomitic natural sand from Elgin, Illinois, a crushed subangular dolomitic limestone from Thornton, Illinois, and a partially crushed, rounded siliceous river gravel from Eau Claire, Wisconsin. Relevant properties are given in table 2.

Table 1. Composition of cement blend.

Item	Percent
C_3S	57
C_2S	17
C_3A	8
C_4AF	8
Alkali (as Na_2O)	0.57
SO_3	2.8
MgO	2.8
Loss on Ignition	1.4
Insoluble Residue	0.2
Specific Gravity	3.11
Blaine Fineness (cm^2/gm)	3680

Table 2. Aggregate properties.

Elgin, Illinois, Fine Aggregate

Grading - % Retained on Sieve Size Indicated							Fineness Modulus	Bulk Specific Gravity - SSD	Absorption - % by wt.
No.	4	8	16	30	50	100	2.72	2.69	1.2
%	1	13	32	53	80	93			

Coarse Aggregate

	Grading - % Retained on Sieve Size Indicated			Bulk Specific Gravity - SSD	Absorption - % by wt.
	3/4 inch	3/8 inch	No. 4		
Thornton Stone	0	50	100	2.73	1.6
Eau Claire Gravel	0	50	100	2.64	1.5

The only admixture utilized was an air-entraining agent consisting of a 2 percent solution of neutralized Vinsol resin.

b. Mix Designs

Three mix designs were used in this study. The combination of these mix designs and two aggregate types lead to the six nominal mixtures summarized in table 3.

Data published by the Portland Cement Association[26] indicate that cement factors currently used by State highway agencies range from a low of 470 lb/yd^3 (278 kg/m^3) to a high of 630 lb/yd^3 (374 kg/m^3), with a mean

Table 3. Mix summary.

Mix No.	Cement Content (lb/yd^3)1/	Air Content (%)	Coarse Aggregate
1	520	5-7	Thornton Limestone
2	610	5-7	Thornton Limestone
3	610	7-9	Thornton Limestone
4	520	5-7	Eau Claire Gravel
5	610	5-7	Eau Claire Gravel
6	610	7-9	Eau Claire Gravel

Slump = 1 to 2 inches (25 to 50 mm)
1/ $kg/m^3 = lb/yd^3 \times 0.5933$

of 556 lb/yd^3 (330 kg/m^3). These low and high points, however, represent only 1 State each. More practical upper and lower limits would be represented by levels of [redacted] In a recent survey of air content specifications reported by Whiting,[27] it was found that while many States retained conventional ranges such as 4 to 7 percent, a number of States were accepting air contents as high as 9 to 10 percent, partly in response to appreciation for higher air content requirements in severe freeze-thaw and deicing environments. For this reason, two air content ranges were chosen for the 610 lb/yd^3 (360 kg/m^3) designs, that is, 5 to 7 percent and 7 to 9 percent. Slump was nominally held to within the range of 1 to 2 inches (25 to 50 mm) to simulate mixtures typically used in slipform pavement operations.

Mixes as prepared differed in slight respects from the nominal designs given in table 3 due to effect of measured air contents on yield. Actual mix analyses, including measured values of slump and air content for each batch, are given in appendix A.

c. Batching and Mixing

Coarse aggregate was weighed and then inundated with water in a closed container 18 to 24 h prior to mixing. Immediately before mixing, a measured amount of water was drained from the container, such that the water remaining (which was subsequently placed into a separate container) would satisfy the absorption of the aggregate plus the net amount of water required for the batch. Fine aggregate was weighed and batched in a moist condition. All mixing was carried out in a 1.75-ft^3 (0.05 m^3) counter-current pan mixer. Charging sequence was coarse aggregate, cement, sand, and the remainder of the mixing water.

A mix cycle consisting of 3-min initial mixing, 3-min rest, and 2-min final mixing, as specified in ASTM Designation: C 192 (Making and Curing Concrete Test Specimens in the Laboratory), was followed. Air-entraining agent was added after an initial 20 sec of mixing. Immediately following the final 2-min mix period, the concrete was tested for slump and air content.

d. Specimen Preparation

Four types of specimens were prepared in this project. These consisted of 6- by 12-inch (15- by 305-mm) cylinders for determination of compressive strength, 3- by 3- by 11.25-inch (76- by 76- by 286-mm) prisms for freeze-thaw testing, 2- by 4-inch (50- by 102-mm) discs for determination of rapid chloride permeability, and concrete cubes for determination of bond to reinforcing steel. These latter specimens were cast as 6-inch (152-mm) cubes with a centrally located No. 6 (3/4-inch, 19-mm) deformed 60-ksi (414 MPa) reinforcing bar located in a horizontal position during casting. Total length of the reinforcing bar was approximately 38 inches (965 mm) with 0.375 inches (10 mm) of the bar protruding from the opposite end of the cast specimen.

The first set of specimens cast from each concrete batch was consolidated using procedures recommended in ASTM C 192. This consisted of consolidation by external vibration using a table vibrator with a frequency of 7000 rpm (116 Hz) and an amplitude of 0.004 inches (0.1 mm). Vibration was applied to specimens held firmly (but not rigidly attached) on the vibrating table until the surface of the concrete had become relatively smooth. Cylinders and blocks were placed in two equal layers, prisms and discs in one layer. After filling with concrete, the weights of concrete contained in each specimen were determined so that subsequent calculations of amounts of concrete needed for additional sets could be made.

Knowing the initial volumes of each mold the amounts of concrete needed to fill 92 and 85 percent of the mold volume were determined for the second and third sets. These amounts of concrete were then weighed into each mold. It was not possible to consolidate these specimens by vibration as the slightest amount of vibration would result in densification above the desired target values. In lieu of vibration, the concrete was hand placed into the molds with only limited manipulation required to achieve the necessary target values.

The final (fourth) set of specimens was designed to represent a concrete which had been overconsolidated. Due to the low slump, good quality aggregates, and proper mix designs, however, this objective proved difficult to achieve. Extended periods of vibration using a "spud" internal vibrator did result in segregation of material in the immediate vicinity of the vibrator, but this was felt to be too heterogeneous to be used in test specimens. A vibration time of 5 min on the table vibrator was finally chosen, it being determined that this amount of vibration led to incipient segregation of coarse aggregate from the mixture and would probably also represent an extreme limit for overvibration in the field of a mix of similar design and quality.

All specimens were finished with a wooden float and placed under moist burlap and polyethylene sheeting for a 24-h period. Molds were then stripped and the castings placed in a fog room at 73 °F (23 °C) for the selected curing periods. Cylinders and cubes were cured for 28 days. Prisms were cured for 14 days (in saturated limewater). Discs were moist cured for 14 days, air dried 14 days, then tested.

Prior to testing, actual bulk densities of all specimens were determined by direct measurement of specimen volume and weight (in air). Percent consolidation was then determined as the ratios of densities of specimens in sets 2, 3, and 4 to the average of companion specimens in set No. 1. Due to such factors as: consolidation due to movement of molds into position for curing, shrinkage upon set, and slight variations in dimensions of molds used, actual percent consolidation deviated from nominally chosen values. Average standard deviation of the difference from nominal values was 2.5 lb/ft^3 (40 kg/m^3) for all specimens.

e. Apparatus and Techniques

All of the techniques used in this program followed standard test methods. Details concerning the apparatus and test procedures can be found in the appropriate standards. Only brief descriptions are included here.

(1) Compressive Strength. The 6- by 12-in (150- by 305-mm) cylinders were capped with a high strength sulfur-based compound and tested using procedures described in ASTM Designation: C 39 (Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens).

(2) Bond of Concrete to Reinforcing Steel. The equipment and techniques used followed those described in ASTM Designation: C 234 (Standard Test Method for Comparing Concretes on the Basis of the Bond Developed with Reinforcing Steel). The 6-inch (150 mm) cube with protruding No. 6 (3/4-inch, 19-mm) reinforcing bar was mounted in a 60,000-lb (27.21 Mg) universal testing machine in series with a 50-ksi (345 MPa) load cell as shown in figure 2. Movement of the loaded end of the reinforcing bar relative to the concrete cube was measured using direct current differential transformers (DCDT) mounted on opposite faces of the cube (figure 3). Load was applied at a rate of 5000 lb/min (22 kN/min). Signals from the load cell and the average signal from the DCDT's were amplified, scaled, and displayed on an X-Y plotter. The deflection data were then corrected for elongation of that portion of the bar between the bearing surface of the concrete and the point of attachment of the strain yoke, yielding true loaded-end slip values. Load data were transformed into stress by taking the surface area of the embedded bar as 14.14 $in.^2$ ($9.12x10^{-3}$ m^2). Critical bond stress was taken either as the stress level at which slip reached a value of 0.01 inch (0.25 mm), or the maximum stress level if pullout occurred prior to a slip of 0.01 inch (0.25 mm).

(3) Resistance to Freezing and Thawing. The equipment and techniques followed those described in ASTM Designation: C 666 (Standard Test Method for Resistance of Concrete to Rapid Freezing and Thawing). Procedure A, Rapid Freezing and Thawing in Water, was used. Testing of the 3- by 3- by 11.25-inch (76- by 76- by 286-mm) prisms was initiated after 14 days of curing. In those cases where the number of specimens due for testing exceeded capacity of the test freezer, the specimens were placed in 4-mil (0.1-mm) thick polyethylene bags and placed in an air-freezer at 0 °F (-18 °C) until sufficient capacity was available for testing. The specimens were then thawed in water overnight at 40 °F (4 °C) and placed under test the following day.

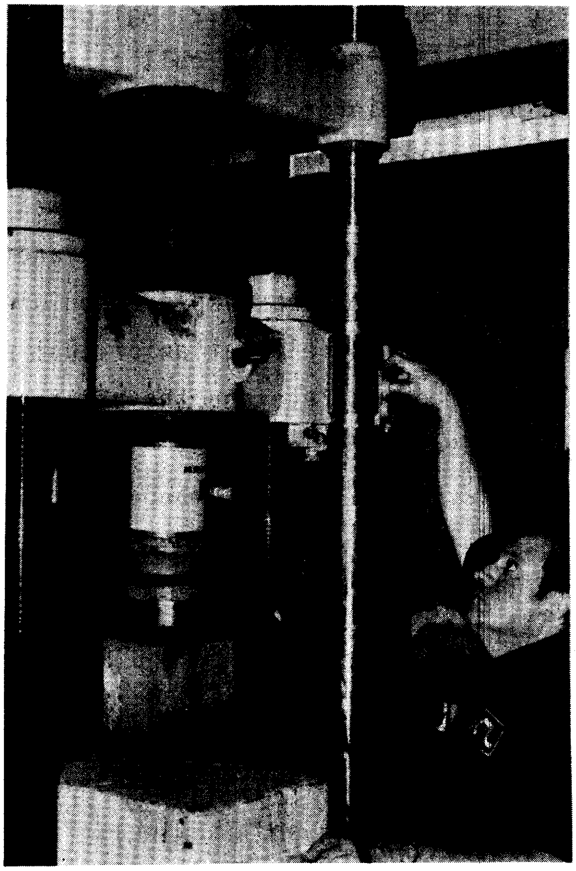


Figure 2. Bond pullout specimen assembly being prepared for test.

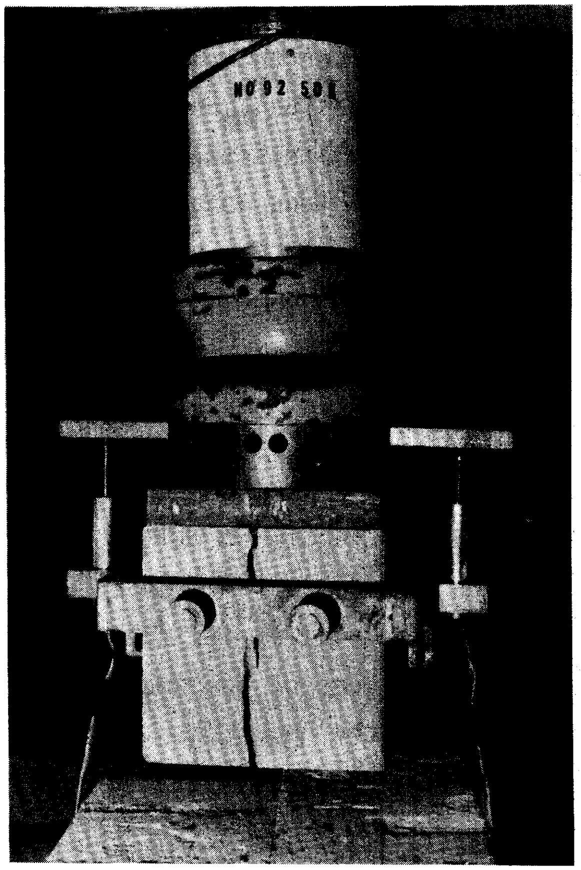


Figure 3. Slip measurement device mounted on test specimen.

(4) Rapid Chloride Permeability. The equipment and techniques followed those described in AASHTO Designation: T 277 (Interim Method of Test for Rapid Determination of the Chloride Permeability of Concrete). The discs were placed in air after 14 days of moist curing and their periphery coated with epoxy after 7 days of drying. After an additional 7 days of drying, testing was initiated using equipment shown in figure 4. The specimens were vacuum saturated with tapwater, then sealed between two halves of an acrylic cell with silicone rubber. Sixty volts dc was applied across the faces of the cell, the positive pole being immersed in 0.3N NaOH, the negative pole immersed in 3.0 percent NaCl solution. Current was monitored over a 6-h period. At the end of this time the current (in amperes) was integrated over time to obtain the amount of charge passed (in coulombs). Previous studies[28] have shown a good correlation between charge passed in this test and long-term penetration of chloride ions into concrete.

3. Discussion of Test Results

a. Appearance of Specimens

A photograph of 6- by 12-inch (150- by 305-mm) cylinders cast at four levels of consolidation from Mix No. 2 is given in figure 5. The lack of consolidation is obvious in the two rightmost cylinders. While not apparent in the photograph taken of the outside surface, the overconsolidated cylinder showed incipient segregation of material at its top when it was cut longitudinally (figure 6). Other types of specimens, i.e. prisms, cubes, and discs, were of similar appearance, the underconsolidated specimens being of an obviously porous and honeycombed character.

b. Compressive Strength

Compressive strength results (after 28 days of moist curing) for all six mixtures are given in table 4. Compressive strength is a strong function of percent consolidation. Even when consolidation decreases by only a little over 10 percent, in most cases strength decreases of 3,000 to 4,000 psi (20 to 28 MPa) are experienced.

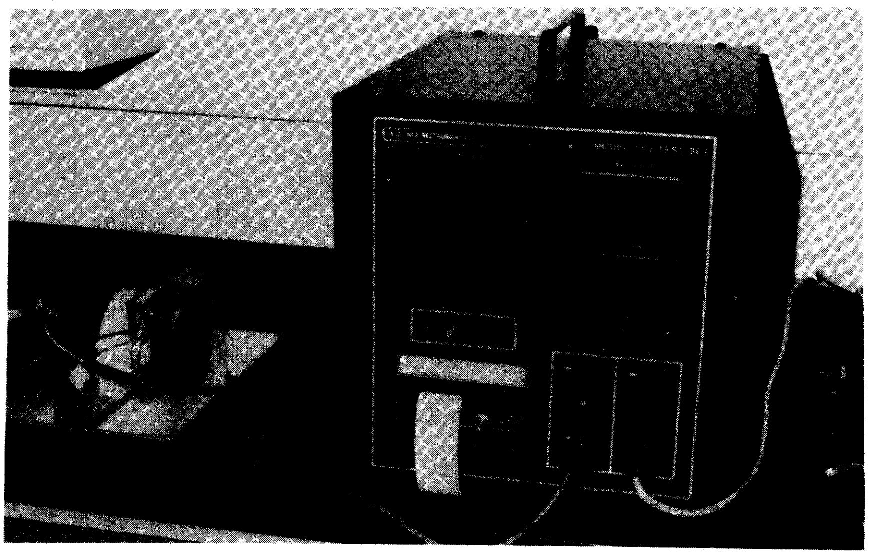


Figure 4. Rapid chloride permeability test apparatus.

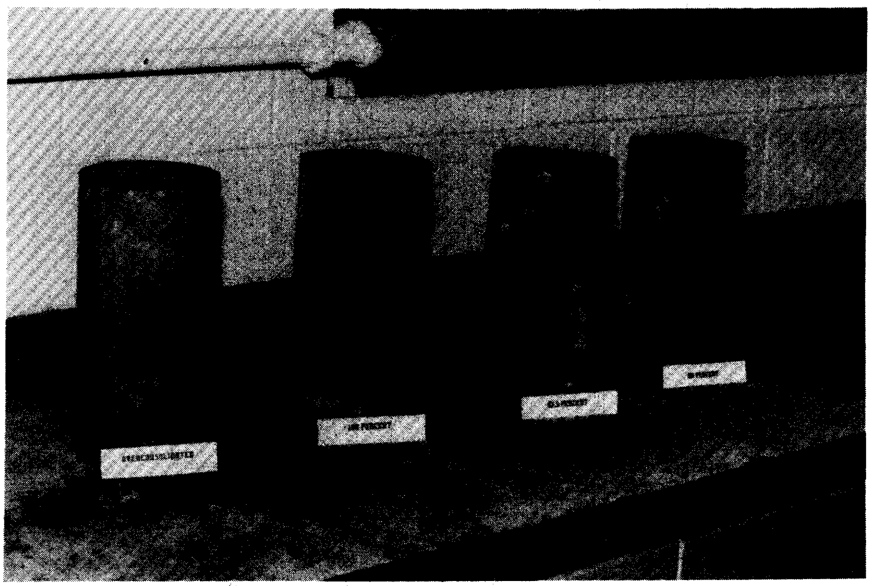


Figure 5. Appearance of cylindrical test specimens prior to testing.

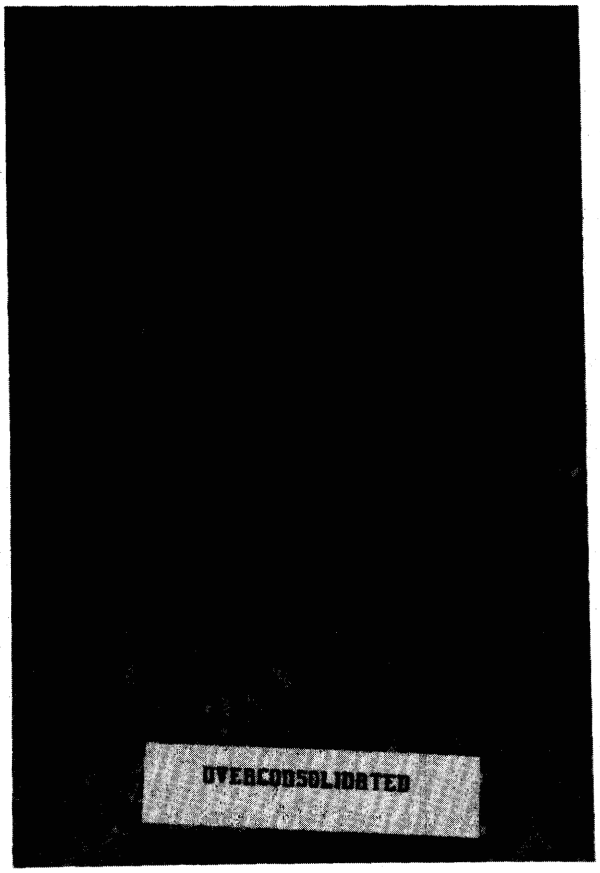


Figure 6. Longitudinal section taken through overconsolidated test cylinder.

Table 4. Compressive strength.

Mix No.	Aggregate	Nominal Cement Content (lb/yd^3)[1]	Air Content[2] (%)	Percent Consolidation[3]	Compressive Strength[4] (psi)[5]
1	Limestone	520	6.6	101.0	4480
				100	4692
				93.3	2655
				87.6	1569
2		610	5.5	101.7	6122
				100	5911
				96.5	4396
				88.0	1863
3		610	8.6	101.4	4932
				100	4930
				88.7	1867
				84.2	1124
4	Gravel	520	6.5	98.5	4202
				100	4573
				91.3	2127
				85.1	1410
5		610	5.4	100.6	5665
				100	5454
				92.6	2386
				87.9	2141
6		610	7.4	99.7	4839
				100	4468
				92.3	2210
				87.4	1818

1. kg/m^3 = lb/yd^3 x 0.5933.
2. Average air content for all batches cast from mix.
3. Average percent consolidation for set of 3 cylinders.
4. Average compressive strength for set of 3 cylinders.
5. MPa = psi x 6.895 x 10^{-3}.

This relationship is graphically illustrated in figure 7 for the limestone mixtures. Here, the reduction in strength (expressed as a percentage of the strength of specimens given normal consolidation effort) is plotted versus percent consolidation. [redacted]. This trend is very similar for the three separate mixtures tested, demonstrating that, at least [redacted] In addition, data developed by Glanville et al., Kaplan and Stewart were averaged and included in figure 7.[5,6,7] It can be seen that the data developed in this study closely overlap those of the previous investigators. Similar relationships are presented in figure 8 for gravel mixtures. Again, all three mixtures show comparable behavior and agree closely with previous data. Finally, comparison of figure 7 with figure 8 [redacted]

c. Critical Bond Stress

Critical bond stress results for the mixtures tested are given in table 5. [redacted] Failure mode in the test specimens changes from one of longitudinal splitting brought about by stress transfer from the bar to concrete, to one in which the bar is simply pulled out of the block with little buildup of frictional restraint, due to the poor bonding between bar and concrete. Bond stress is reduced about 10 to 15 percent for each percent decrease in consolidation. This is close to the value of 10 percent for each percentage of increased void space cited by Hognestad and Siess,[12] in their study of the effect of air content on bond of reinforcing steel. Reduction in bond stress, then, may be more than double the percentage reduction in compressive strength for a given percentage reduction in consolidation. [redacted]

These relationships are illustrated in figure 9 for limestone mixtures and figure 10 for gravel mixtures. While differences between mixtures with respect to the relationships shown in figures 9 and 10 are apparent, they are not consistent, and most likely are attributable to variations in the experimental data.

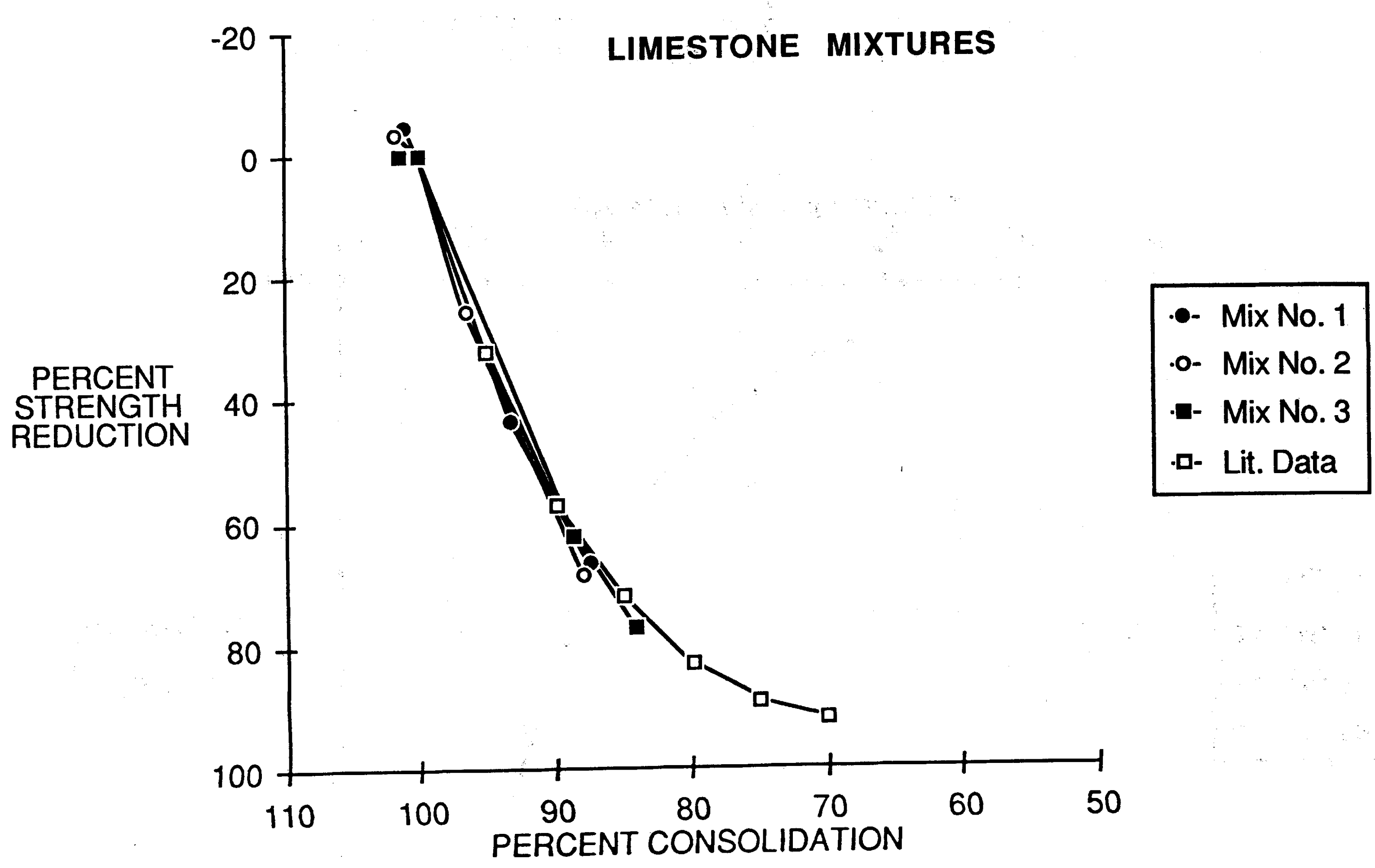


Figure 7. Relationship between percent consolidation and percent reduction in 28-day compressive strength for limestone mixtures.

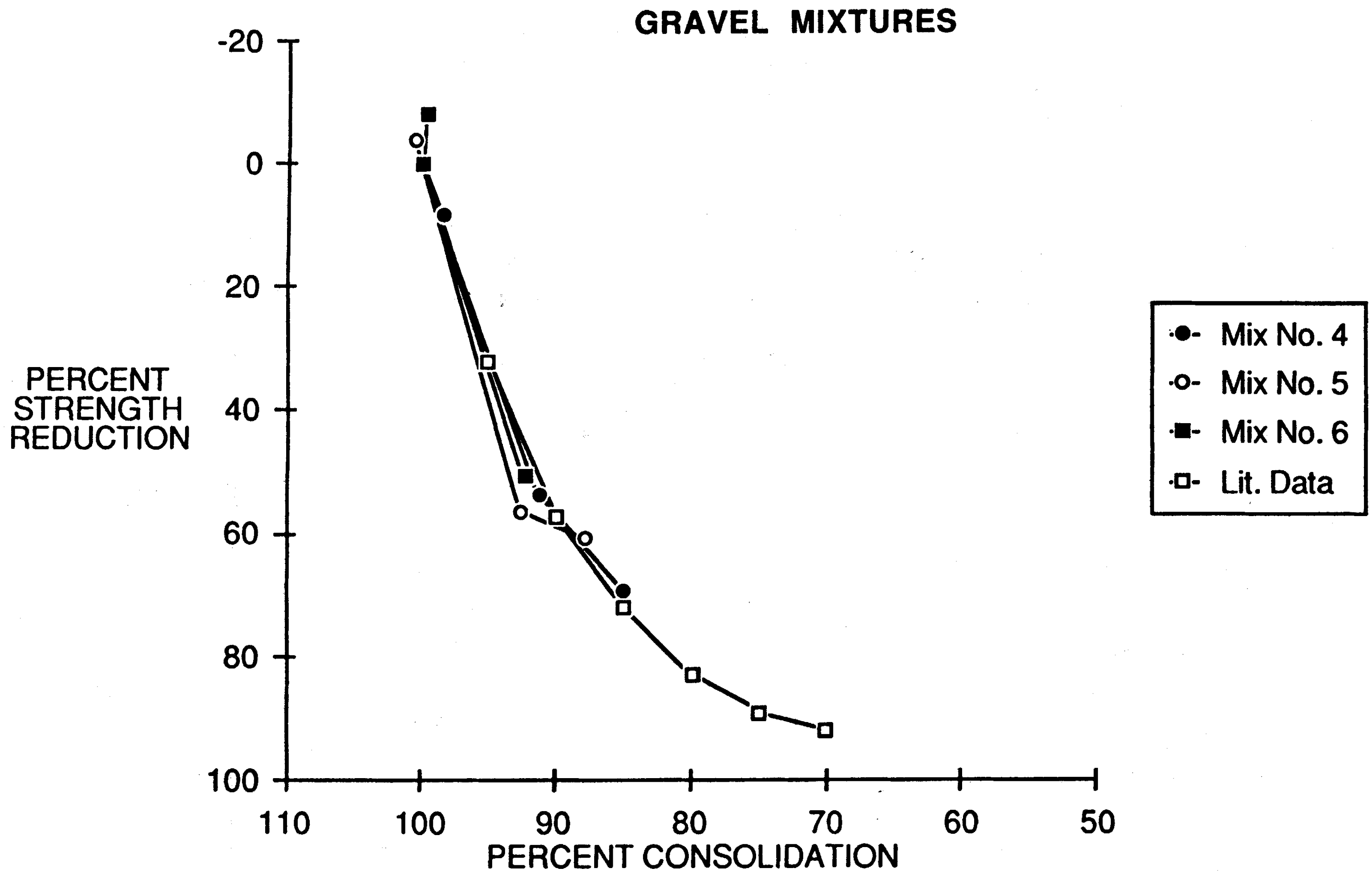


Figure 8. Relationship between percent consolidation and percent reduction in 28-day compressive strength for gravel mixtures.

Table 5. Critical bond stress.

Mix No.	Aggregate	Nominal Cement Content (lb/yd^3)[1]	Air Content[2] (%)	Percent Consolidation[3]	Critical Bond Stress[4] (psi)[5]
1	Limestone	520	5.8	102.7	1239
				100	942
				92.3	397
				88.0	250
2		610	5.9	102.6	1285
				100	1144
				94.6	392
				91.1	245
3		610	8.6	100.0	1042
				100	1019
				91.5	281
				88.6	192
4	Gravel	520	6.5	102.3	1193
				100	1279
				94.9	299
				89.5	132
5		610	5.4	101.9	1431
				100	1263
				91.9	365
				86.8	274
6		610	7.4	100.1	1055
				100	1031
				92.5	436
				85.8	108

1. $kg/m^3 = lb/yd^3 \times 0.5933$.
2. Average air content for all batches cast from mix.
3. Average percent consolidation for set of 3 cubes.
4. Average critical bond stress for set of 3 cubes.
5. $MPa = psi \times 6.895 \times 10^{-3}$.

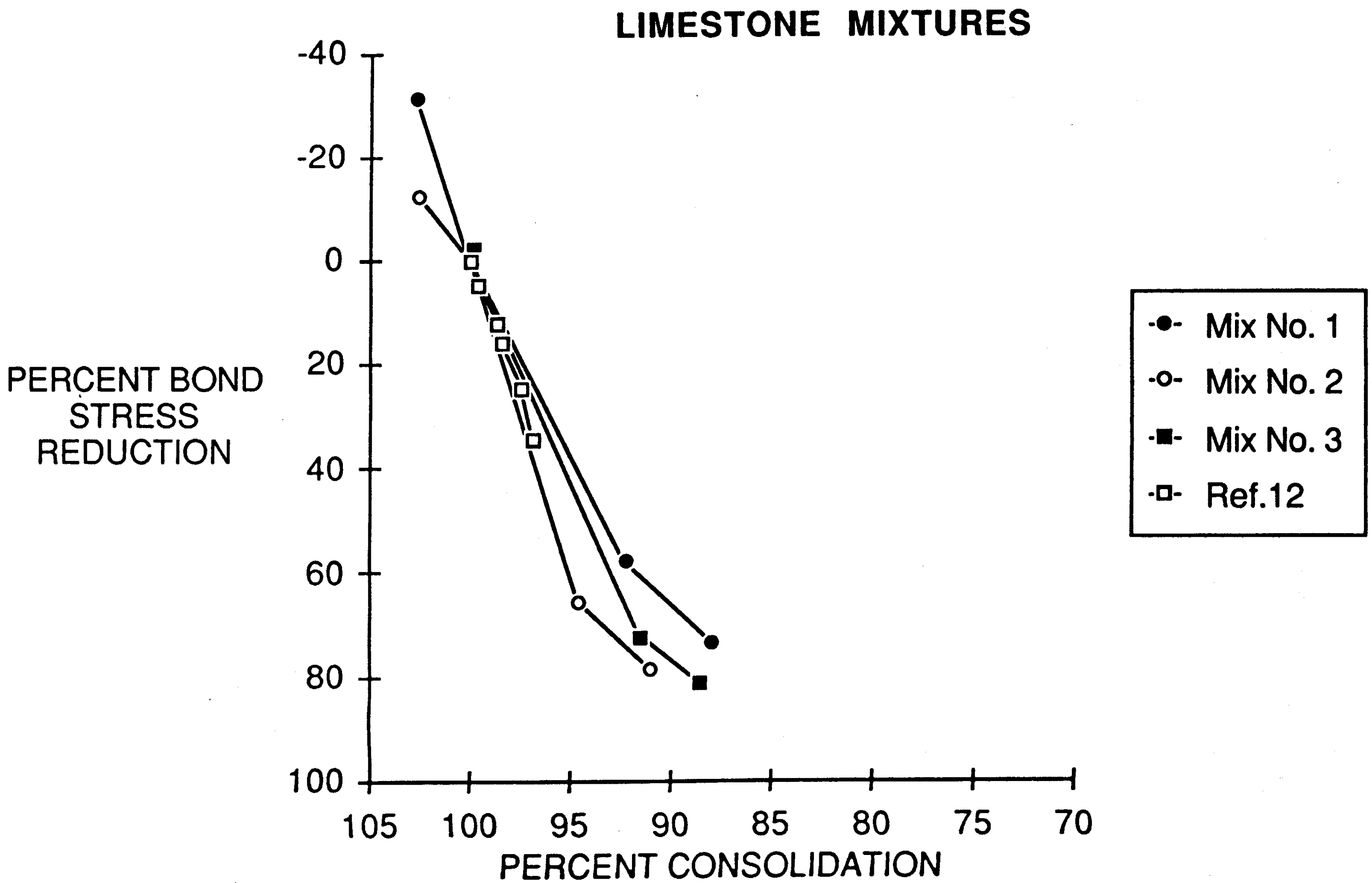


Figure 9. Relationship between percent consolidation and percent reduction in critical bond stress for limestone mixtures.

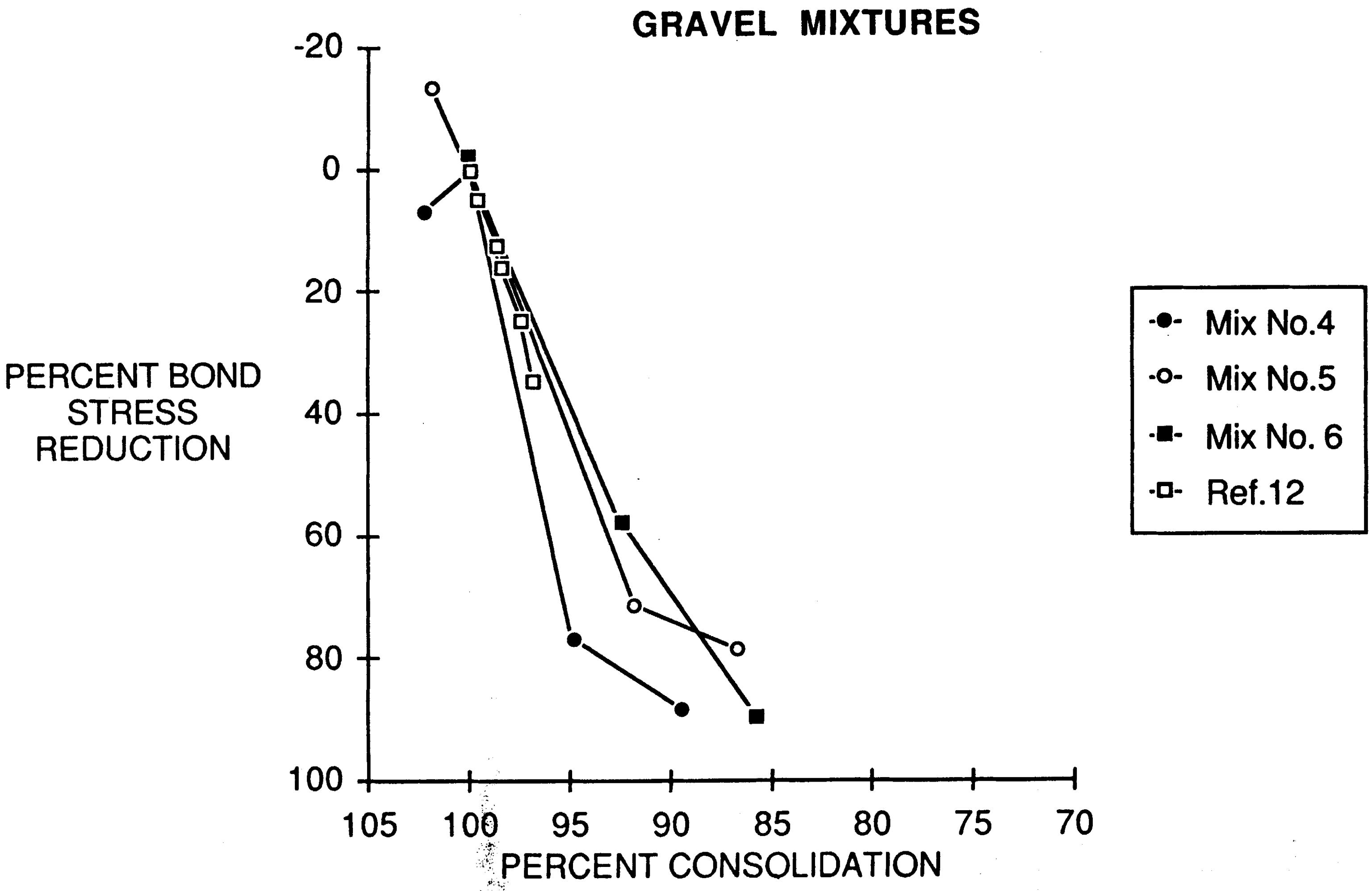


Figure 10. Relationship between percent consolidation and percent reduction in critical bond stress for gravel mixtures.

[redacted] greater than the increase in compressive strength experienced for comparable test cylinders. It is likely that this increase in bond stress results from expulsion of entrained and entrapped air from the test specimens, which allows the concrete to more completely consolidate around the bar and therefore develop improved bond characteristics. This may be analogous to the beneficial effects of revibration on bond reported by some authors.[11]

d. Resistance to Freezing and Thawing

Results are presented in table 6. For the limestone aggregate mixtures [redacted] ng. All values of the relative dynamic modulus of elasticity (RDM) at 300 cycles exceeded 90 percent, and weight losses were all less than 3 percent.

For the gravel mixtures only those concretes at the lowest level of consolidation (Mix No. 6, 86 percent consolidation) showed appreciable deterioration based on reduction of RDM. However, even this reduction is not overly severe considering the harshness of the test, and weight losses were less then 5 percent for these specimens.

e. Permeability to Chloride Ion

Results of testing for permeability to chloride ion using the AASHTO Rapid Test procedure are presented in table 7. [redacted] As this test is of a semiquantitative nature, it was felt that graphical presentation could best be accomplished in a bar chart format, where mixtures could be compared at equal percent consolidation. To accomplish this, plots of the data were constructed, and the values of charge passed were obtained at preselected levels of 102, 100, 96 and 92 percent consolidation.

Such an illustration for the limestone mixtures is presented as figure 11. For any given mixture, the lowest permeability is exhibited by the "overconsolidated" specimens. As with the increases in bond strength for companion specimens, this effect may be attributable to expulsion of entrained and entrapped air during the extended vibration given these specimens.

Table 6. Freeze-thaw results.

Mix No.	Aggregate	Nominal Cement Content (lb/yd^3)[1]	Air Content[2] (%)	Percent Consolidation[3]	Weight Change[4] (%)	RDM[5] (%)
1	Limestone	520	6.2	102.3	-0.5	97
				100.0	-0.7	103
				94.0	-0.8	97
				87.3	-1.0	96
2		610	5.6	102.0	-0.2	101
				100.0	0.1	100
				95.7	-0.8	98
				88.8	-1.9	91
3		610	8.0	102.8	0.3	101
				100.0	-0.3	100
				96.3	-1.1	102
				90.0	-2.8	94
4	Gravel	520	6.3	103.6	0.0	100
				100.0	0.0	100
				95.3	-0.6	97
				90.6	-0.8	96
5		610	5.4	103.4	0.1	100
				100.0	0.1	100
				95.6	-1.0	98
				89.4	-2.1	80
6		610	7.3	101.5	0.1	100
				100.0	0.2	97
				94.0	-2.4	91
				86.2	-4.8	74

1. kg/m^3 = lb/yd^3 x 0.5933.
2. Average air content for all batches cast from mix.
3. Average percent consolidation for set of 6 prisms.
4. Average weight change for set of 6 prisms.
5. Average relative dynamic modulus (RDM) for set of 6 prisms.

Table 7. Rapid chloride permeability.

Mix No.	Aggregate	Nominal Cement Content (lb/yd^3)[1]	Air Content[2] (%)	Percent Consolidation[3]	Charged Passed[4] (Coulombs)
1	Limestone	520	6.0	101.0	2902
				100.0	3288
				95.9	4585
				90.6	8823
2		610	5.3	102.1	1880
				100.0	2530
				93.3	3264
				86.8	7289
3		610	8.0	102.9	2791
				100.0	3754
				97.9	3865
				92.8	8411
4	Gravel	520	5.9	102.6	1535
				100.0	1804
				94.4	2383
				90.3	5138
5		610	5.3	101.0	1609
				100.0	1758
				95.4	2383
				89.0	4078
6		610	7.3	101.2	1768
				100.0	1828
				93.9	3166
				89.0	2629

1. kg/m^3 = lb/yd^3 x 0.5933.
2. Average air content for all batches cast from mix.
3. Average percent consolidation for set of 4 discs.
4. Average charge passed for set of 4 discs.

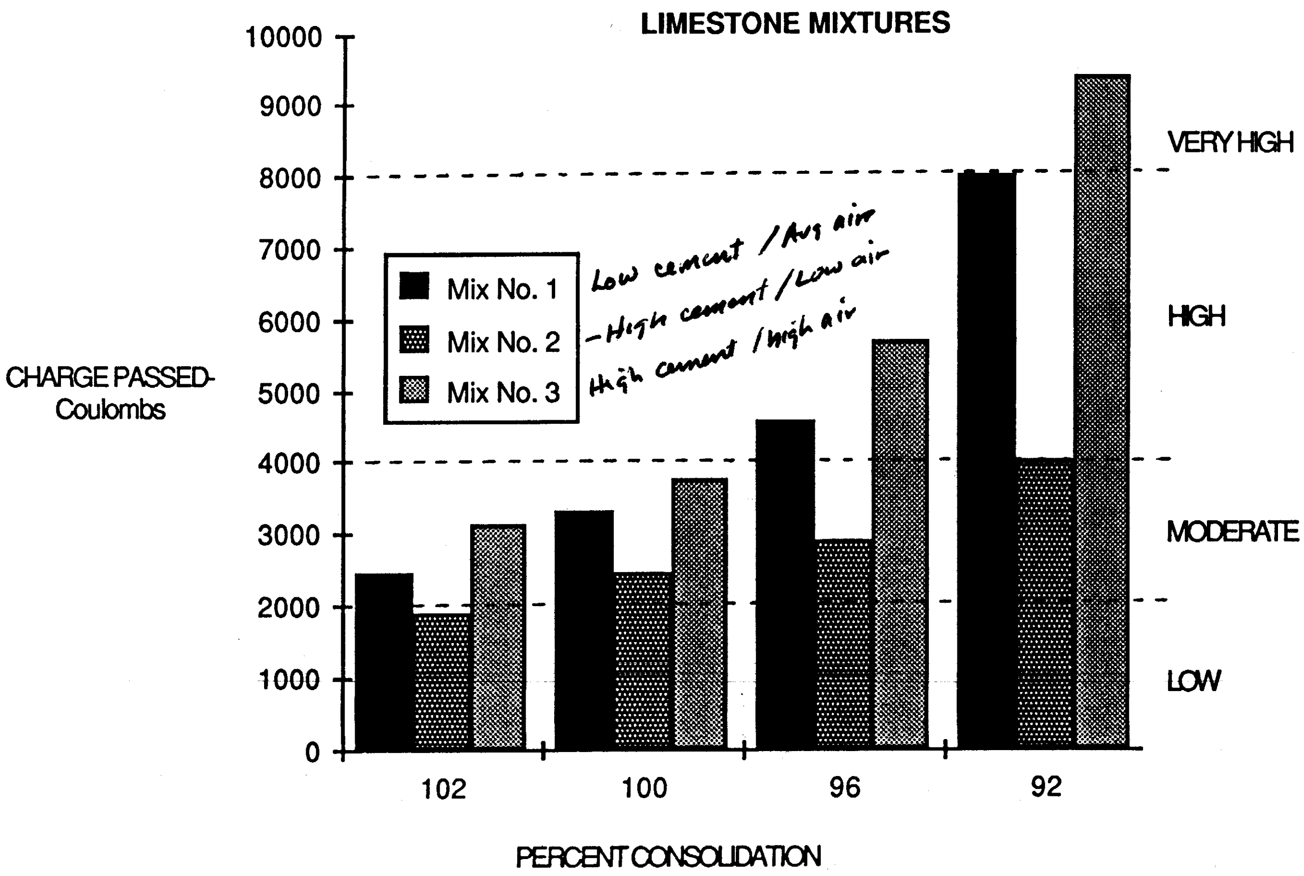


Figure 11. Chloride permeability of limestone mixtures.

Sprinkle has noted that permeability to chloride ion, as measured by this rapid test, does decrease with decreasing total air content.[29] This effect is also seen with respect to mix No. 3 which consistently shows higher permeability than corresponding mix No. 2, which was prepared at lower air content. [redacted] Mix No. 1, prepared at higher w/c ratio (average 0.47) exhibits somewhat lower permeability at all levels of consolidation than mix No. 3, prepared at lower w/c ratio (0.42) because of this air void effect. Mix No. 2, prepared at low w/c ratio (0.42) and air content (5.3 percent) exhibits the lowest permeability, although even for these specimens permeability approaches "high" values at 92 percent consolidation.

For gravel mixtures (figure 12) some of these trends are less apparent. While there still is a general tendency towards increasing permeability with decreasing degree of consolidation, the effects of air content and w/c ratio are less pronounced. In addition, overall permeability levels are considerably lower than those for corresponding limestone mixtures. This may be attributed to the lower w/c ratio used in the gravel mixtures, where the rounded aggregate required less water for a given slump at equal cement content than the angular crushed limestone, difference between the two mixtures being about 0.04 w/c ratio.

4. Statistical Analysis of Test Data

To facilitate a more quantitative understanding of the relationships between degree of consolidation and the concrete properties under study, the experimental data were subjected to statistical analysis. As the majority of specimens tested for freeze-thaw resistance showed insignificant deterioration, these data were not included in this analysis.

As previously noted, it had not been possible to control consolidation exactly at fixed predetermined levels; therefore, an analytical technique which would allow for these variations about the nominal values was selected. The approach chosen was to fit the data to a linearized relationship using least squares regression. Best fit was found for a semilogarithmic relationship of the form

$$\log_{10} Y = A_0 + A_1 X \qquad (2)$$

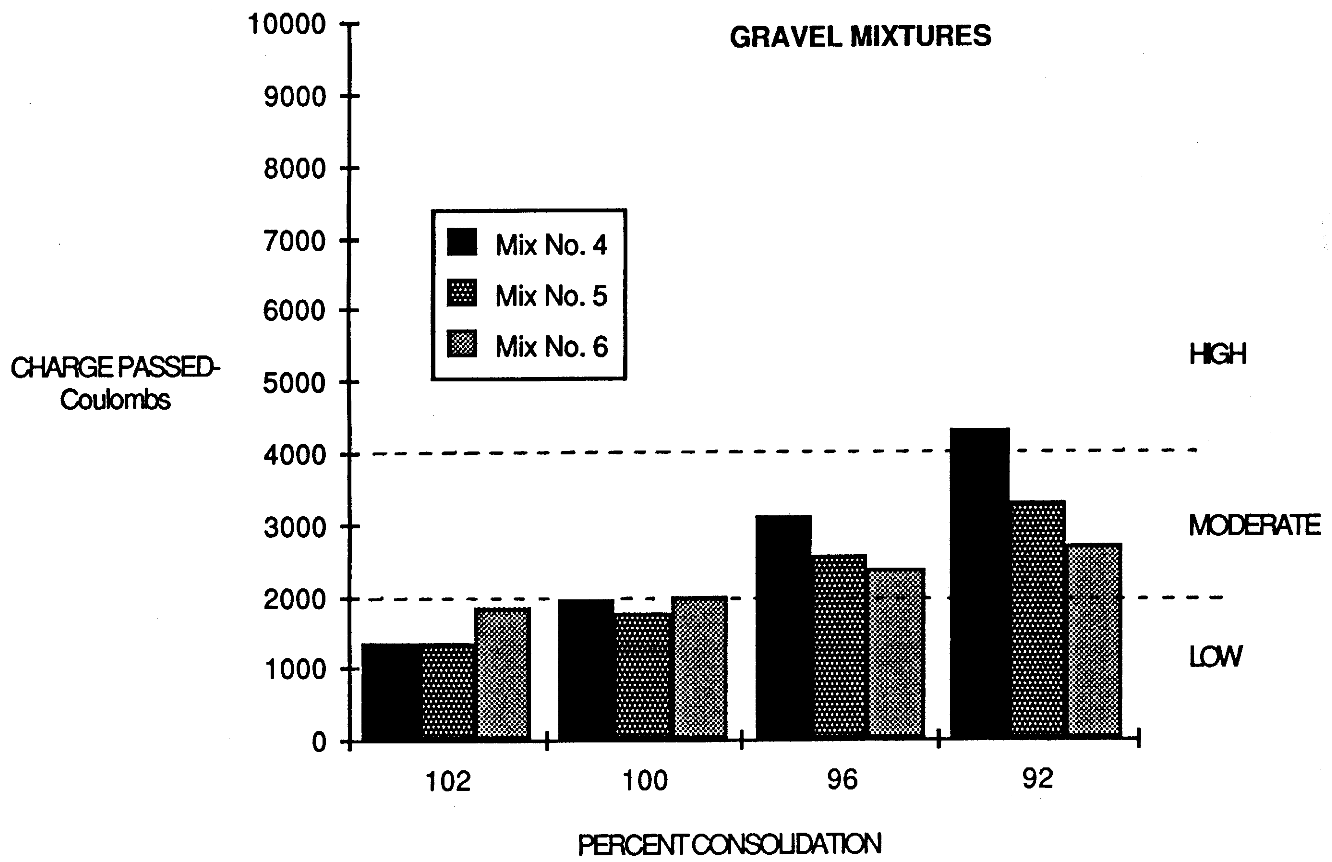


Figure 12. Chloride permeability of gravel mixtures.

where, in the present study:

Y = property of interest (i.e., compressive strength, critical bond stress, or charge passed)

X = percent consolidation

A_0 = constant (Y-intercept)

A_1 = constant (slope of line)

After fitting data from each mixture to equation 2, normalized relationships were obtained by expressing the properties in terms of the value of the property at 100 percent consolidation. The values at 100 percent, Y_{100}, were obtained from the predicted values from equation 2. The fractional values at other degrees of consolidation, Y_x, were then obtained by dividing the experimental data by the respective Y_{100} values. These nondimensional Y_x values were then again fitted to the percent consolidation, X, as

$$\log_{10} Y_x = A_0+A_1X \tag{3}$$

where: Y_x = fraction value of property of interest at degree of consolidation x

X = percent consolidation

A_0 = constant (Y-intercept)

A_1 = constant (slope of line)

Because of the normalization employed, A_0 becomes minus 100 times A_1. Equation 3 may then be expressed in a more convenient form as

$$Y_x = 10^{2-[A_1(100-x)]} \tag{4}$$

Values of the slope constant (A_1), correlation coefficient (R) and standard errors of estimate (in absolute and percentage units) are given in table 8. In most cases, reasonably good fits were obtained, as reflected in values of R.

The slope constant (A_1) can be used as a measure of the behavior of each property with respect to the variables inherent in each mixture. As cement content increases (mixes 2 and 5 versus 1 and 4), A_1 generally increases for compressive strength and bond stress, indicating a greater sensitivity to degree of consolidation for mixtures of higher cement content. For chloride permeability the converse is true: as cement content increases, the concrete

Table 8. Statistical parameters.

Variable	Mix No.	A_1	R	Std. Error Absolute (psi)[1]	Std. Error Relative (%)
Compressive Strength	1	.0332	0.899	785	19.2
	2	.0390	0.972	560	10.2
	3	.0398	0.979	412	8.8
	4	.0343	0.973	334	7.5
	5	.0356	0.958	361	7.3
	6	.0351	0.929	596	13.2
Critical Bond Stress	1	.0492	0.923	191	18.6
	2	.0703	0.839	261	29.9
	3	.0676	0.941	276	28.4
	4	.0781	0.777	297	37.2
	5	.0588	0.881	347	30.9
	6	.0772	0.933	223	19.7
				(Coulombs)	
Chloride Permeability	1	-.0404	0.907	901	25.5
	2	-.0347	0.949	1062	40.4
	3	-.0429	0.824	1962	53.9
	4	-.0403	0.593	507	24.6
	5	-.0314	0.917	613	32.7
	6	-.0158	0.693	576	27.6

1. MPa = psi x 6.895×10^{-3}

is less sensitive to variations in consolidation. In the case of air content comparisons (mixes 3 and 6 versus 2 and 5) there are no consistent trends in A_1. This indicates that mixtures of similar design will show equal sensitivity to consolidation irrespective of their relative air content.

While treatment of each separate mixture is useful for comparison of mix design variables, more general predictive relationships can be obtained by analysis of each total set of data relating to compressive strength, bond stress, and permeability. These data were fitted to the nondimensional log relationship, and resultant curves (including 95 percent confidence limits) are shown in figures 13 through 15.

Figure 13, showing the relationship between percent consolidation and compressive strength, can be used as an illustration of the application of these results to specifications. Data tabulated in section 4 of this report indicate that a number of State highway agencies impose a minimum limit of 98 percent consolidation on inplace bridge deck overlay concrete. According to figure 13 this would imply that a mean reduction down to 84 percent of strength would be allowed. However, at least 5 percent of test results could actually be greater than 114 or less than 65 percent of the 100 percent strength level. In addition, variance associated with the method of determining percentage consolidation (usually done using commercial nuclear density gauges) would need to be included in a statistically based end result specification for concrete consolidation.

5. Conclusions Based on Results of Laboratory Investigations

Based on results of this laboratory investigation, the following conclusions may be drawn:

- Degree of consolidation has a strong effect on compressive strength. There is a loss of approximately 30 percent in strength for every 5 percent decrease in consolidation. Mixtures having higher cement contents show a somewhat greater sensitivity to strength loss for a given percentage decrease in consolidation. Type of aggregate used and air content have little effect on this relationship.

- Degree of consolidation has an even more pronounced effect on critical bond stress. Bond stress is reduced by more than 50 percent for a 5 percent decrease in consolidation. Effects of mix variables are less consistent in the case of bond stress, indicating the predominant

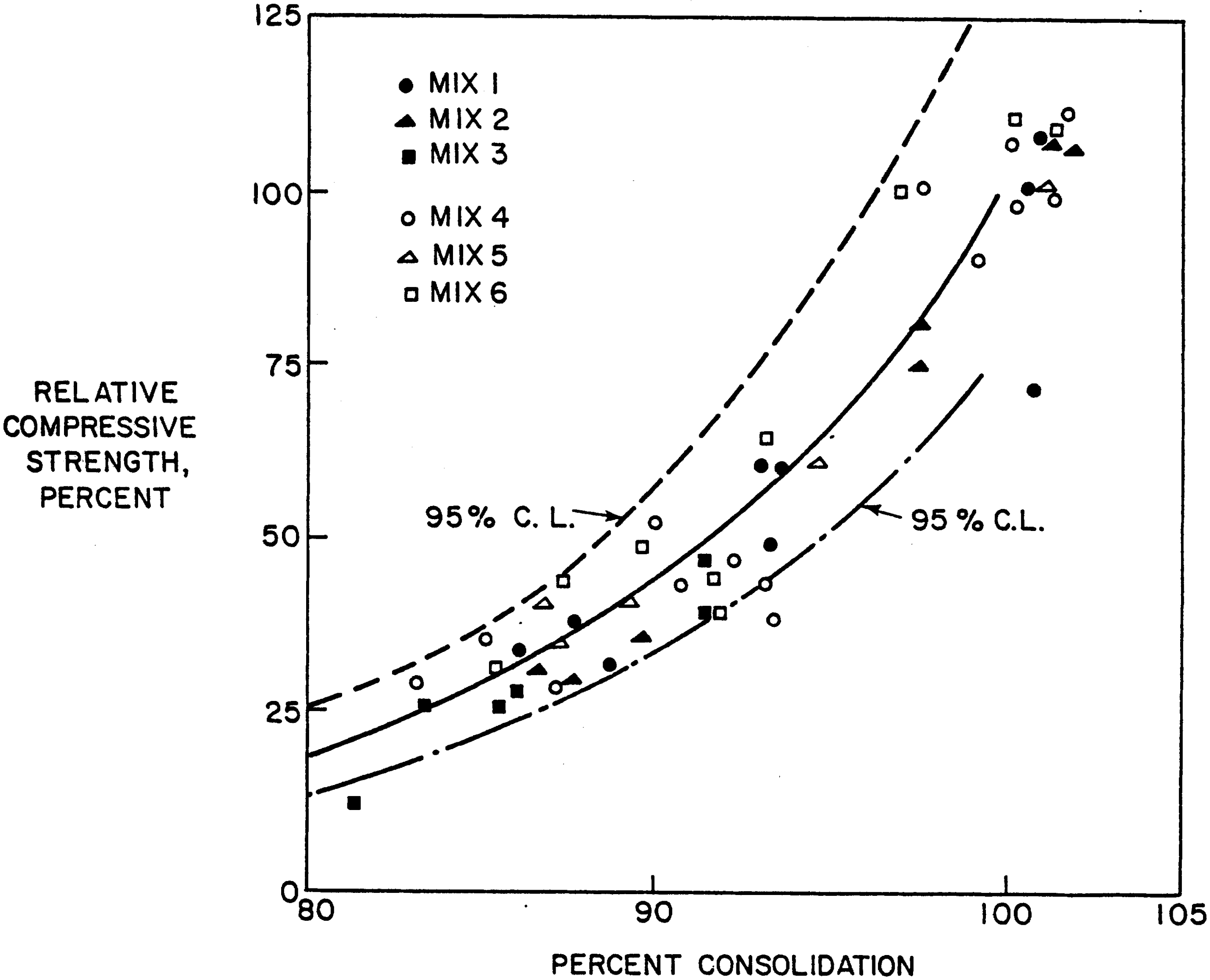


Fig. 13. General relationship between percent consolidation and relative compressive strength - all mixtures.

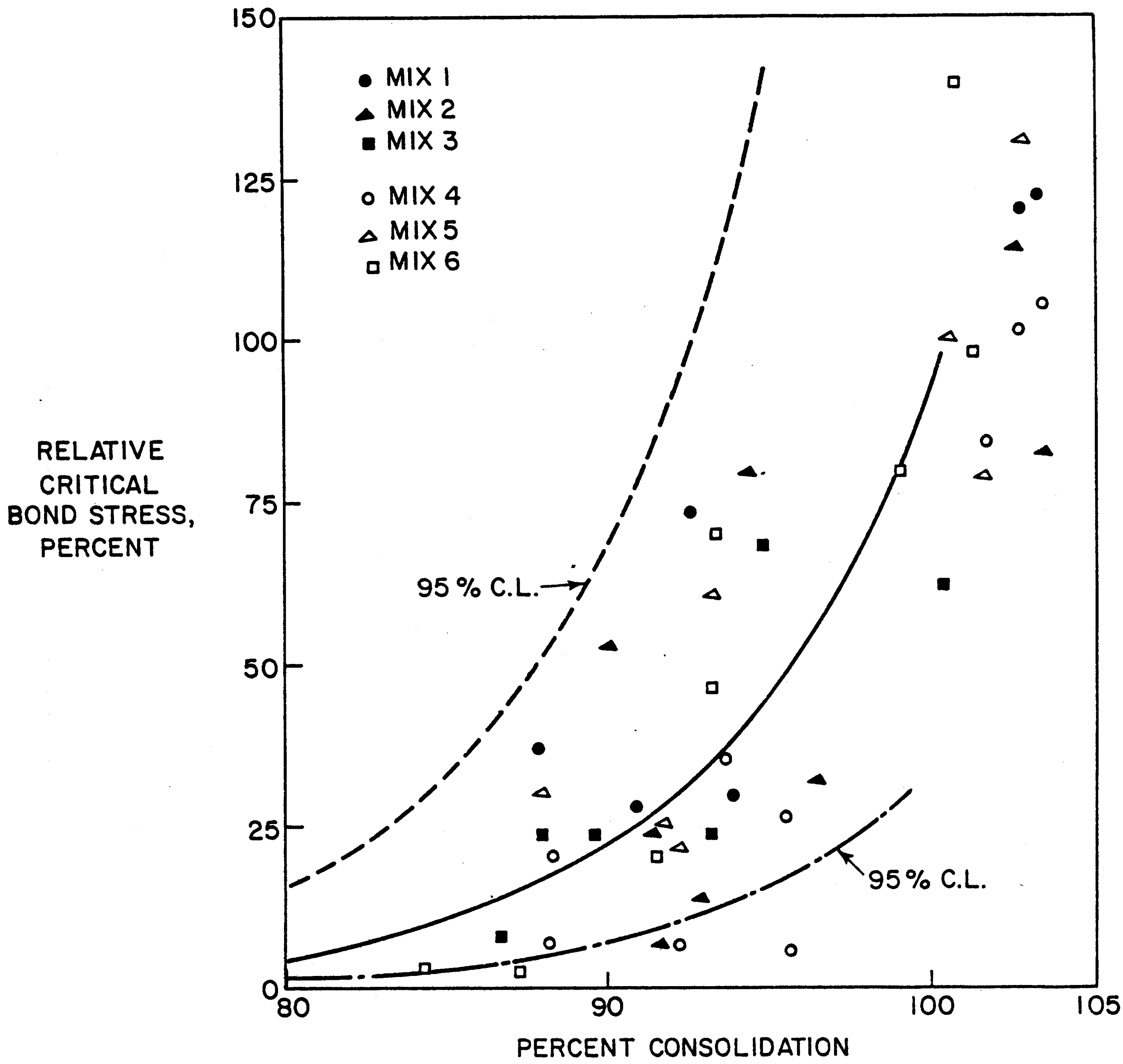


Fig. 14. General relationship between percent consolidation and relative critical bond stress - all mixtures.

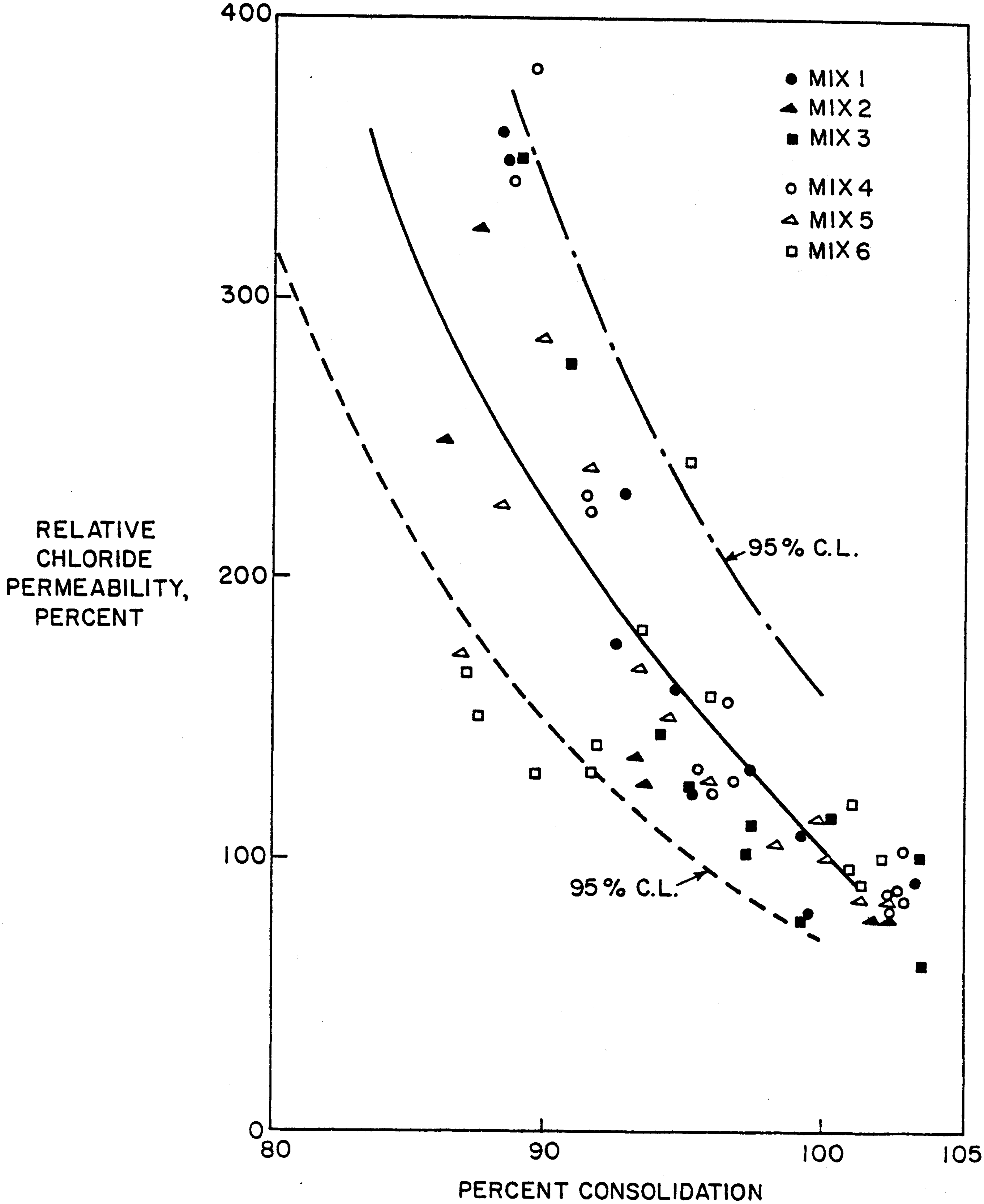


Fig. 15. General relationship between percent consolidation and relative chloride permeability - all mixtures.

effect to be a loss in bond due to increasing void space in the vicinity of the reinforcing steel.

- The effects of degree of consolidation on the freeze-thaw resistance of properly air-entrained concrete are much less than the corresponding effects on strength. Most concretes tested in this series performed satisfactorily in accelerated freezing and thawing tests, irrespective of their degree of consolidation. Limestone mixtures showed somewhat greater resistance to freezing and thawing then did gravel mixtures having similar mix designs.

- Permeability to chloride ions (as measured by the rapid AASHTO technique) increases with a decrease in percent consolidation. The effect is greater in limestone mixtures and in mixtures with high levels of entrained air. The effect becomes more pronounced below 96 percent consolidation.

- In the majority of cases, overconsolidation to the point of incipient segregation results in improvements in all properties tested. Compressive strength increases slightly, bond stress increases by as much as 20 percent, weight loss during freezing and thawing is slightly reduced, and permeability to chloride ion decreases. These effects are most likely due to expulsion of entrapped and some entrained air from the test specimens during extended external vibration.

ASSESSMENT OF THE USE OF NUCLEAR DENSITY GAUGES FOR MONITORING CONSOLIDATION OF CONCRETE

1. Introduction to Nuclear Gauge Technology

The measurement of inplace density of a variety of construction materials has been greatly facilitated through the application of nuclear gauge technology. The release of gamma rays, or photons during decay of certain radioactive nuclei forms the basis for the technique (gamma rays are chargeless electromagnetic radiation which have zero mass and which travel at the speed of light). Gamma radiation of the type used in commercial nuclear gauges interacts with matter primarily by photoelectric absorption and Compton scattering. These processes are utilized in direct transmission and backscatter nuclear density gauges.

Initial applications of nuclear gauges focused on measurement of density of soils during construction of highway subbases. Good comparisons with standard sand-cone techniques were generally obtained.[30,31] A continuous measuring device, termed the "Road Logger," has also been successfully applied to measurement of soil compaction.[32]

Nuclear gauges have also been applied to measurement of compaction of asphaltic concrete. They are used both on full-depth pavements and, more recently, on thin asphalt overlays. Use is increasing in this area.

The primary emphasis in this report is on the use of nuclear gauges for monitoring consolidation of portland cement concrete. As with soils and asphalt, success in early research studies led to adoption of nuclear technology by many State highway agencies for routine measurement. Thorough background investigations were done by the Mississippi Highway Department.[33] Direct transmission gauges were used at depths of 2 inches (50 mm) and 6 inches (150 mm). These were compared with unit weight tests of the plastic concrete (AASHTO-T121). Density measurements were made with the nuclear device and the readings were within ± 2.5 lb/ft^3 (40 kg/m^3) of results of conventional unit weight tests. Federal Highway Administration studies also used direct transmission gauges to monitor fresh concrete with good results in laboratory investigations.[34]

Work done by the Colorado Division of Highways confirmed the value of nuclear density devices for indicating consolidation of fresh concrete immediately behind a slipform paver.[35,36] In their tests, nuclear readings averaged 0.37 lb/ft^3 (5.9 kg/m^3) lower than the average of standard density tests. Results of these and other studies indicate that nuclear gauges have certain advantages and disadvantages for use in measurement of concrete consolidation. To appreciate the reasons for these advantages and disadvantages, the operating characteristics of the various gauge types must be understood and are discussed in subsequent sections.

a. Direct Transmission Gauges

One of the two designs used for static density measurements of concrete in highway construction is the direct transmission gauge. The transmission gauge is characterized by the placement of the source of radiation and the radiation detector so that the portion of the concrete to be measured is between the source and detector. As the density of the concrete increases (the spacing between source and detector must be held constant), the radiation intensity detected decreases. The relationship for intensity (I) versus density can be represented ideally as:

$$I = I_0 \exp(-\mu\rho t) \qquad (5)$$

where: I is the intensity for density ρ;

I_0 is the intensity with no specimen;

μ is the absorption coefficient for the material and radiation used;

ρ is the density of the specimen; and

t is the thickness of the specimen.

The equation above holds only for the circumstances in which radiation that has not interacted in any way with the specimen is detected. Such circumstances usually include either a narrow beam of radiation containing only parallel rays or a radiation detection system that responds only to full energy, i.e., unscattered, photons of the radiation. A fairly well-collimated versus a poorly collimated radiation gauge system is diagrammed in figure 16. If the poorly collimated system is used, the relationship between intensity and density must include a factor related to the detection of scattered radiation that is detected. Equation 5 would then take the following form:

$$I = I_0 B \exp(-\mu\rho t) \qquad (6)$$

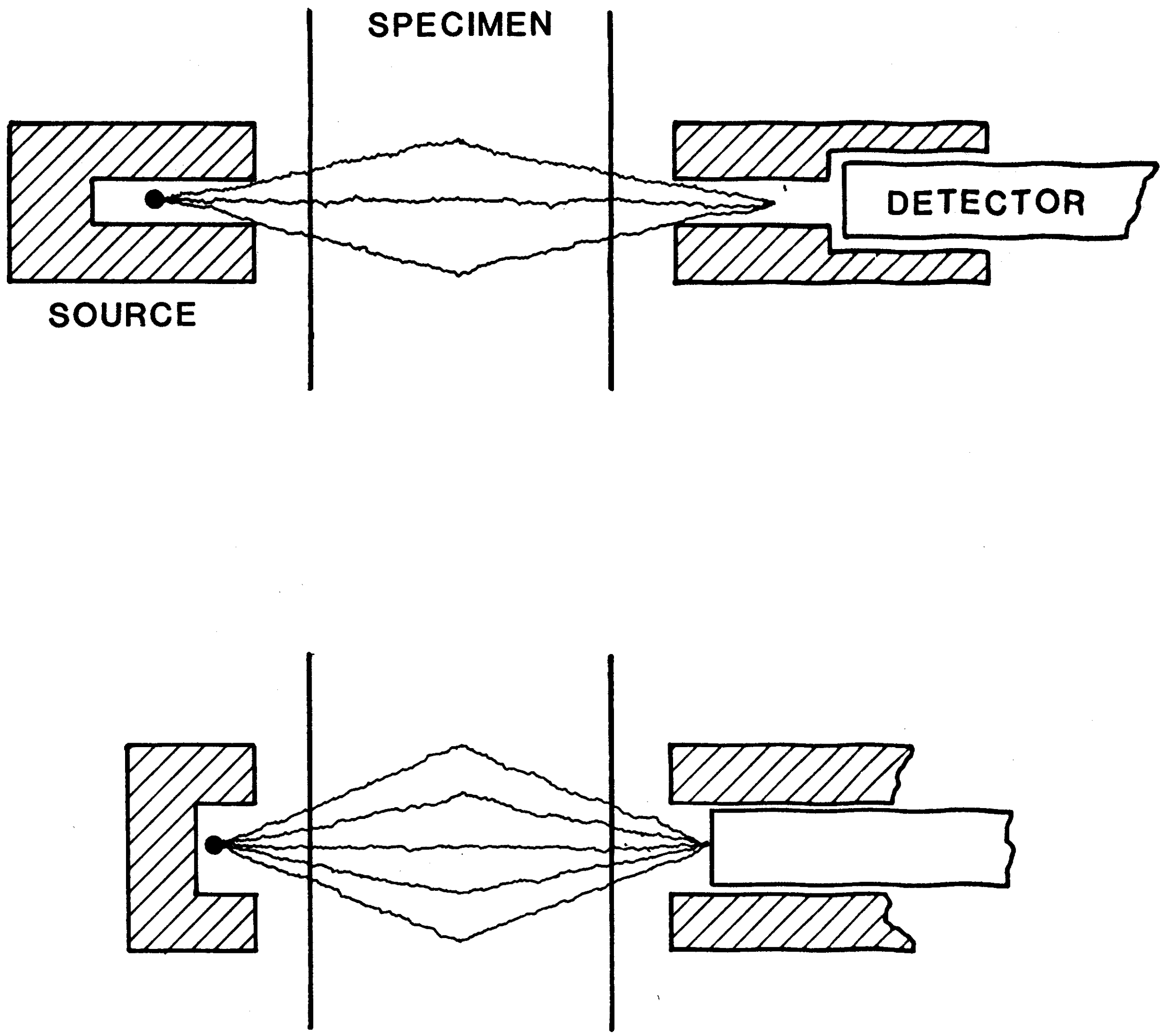


Fig. 16. Well-collimated (top) and poorly collimated (bottom) direct transmission gauges.

where B represents a "build up" factor which must be determined empirically for the system involved.

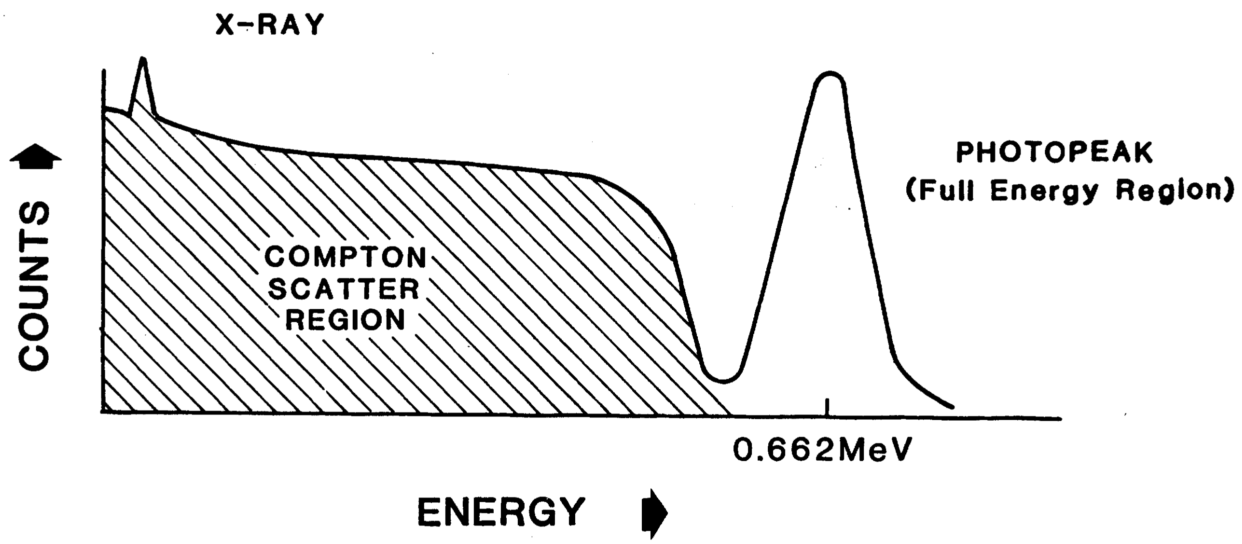
The radiation detectors can be designed to detect only full energy (unscattered) radiation from the source. An example would be the NaI(Tl) scintillation detector used in the CMD. Figure 17 is a gamma energy spectrum obtained by a NaI(Tl) detector showing the sensitivity obtained in the full energy (photopeak) region.

If only full energy gamma photons are detected, a straight line relationship is obtained for the plot of the logarithm of intensity (ln I) vs. density

(ρ) (see figure 18). Also included in figure 18 is the nonlinear relationship obtained for the poorly collimated gauge in which scattered radiation is detected. Note that the slope of the line is less for the case in which scattered radiation is detected, indicating that the sensitivity for measuring specimen density is decreased. Gauges using no collimation and/or a detector system with no energy discrimination will have poorer sensitivity for changes in density than gauges with good collimation and/or energy discrimination.

Transmission gauges using single energy radioisotopic sources (such as Cs-137) in the range 0.3 to 3 MeV (0.48 to 4.8 x 10^{-14} J) also provide density measurements that are relatively unaffected by chemical changes in the specimen. This is true because the absorption coefficient (μ) in equation 5 changes very little for vastly different elements in this energy range. Figure 19 is a graphical representation of the relationship between the absorption coefficient and energy for several materials. Note that between 0.3 and 3 MeV (0.48 and $4.8\text{x}10^{-14}$ J) the absorption coefficient changes very little with energy and there is little difference for the various materials.

The intensity of radiation is measured as the number of events detected in a fixed length of time (count time). Since radioactive decay is a random process, the measurements can be described in statistical terms. The measurements can be represented to have a mean, standard deviation (σ), and confidence level. Of particular value for static measurement gauges is the concept that gauge precision will be related to the number of events detected, which will be related to the count rate of the detection system multiplied by the counting time. Table 9 serves to indicate the importance of these factors.



$J = MeV \times 1.6 \times 10^{-13}$

Fig. 17. Cesiuim-137 energy spectrum using NaI(Tl) detector.

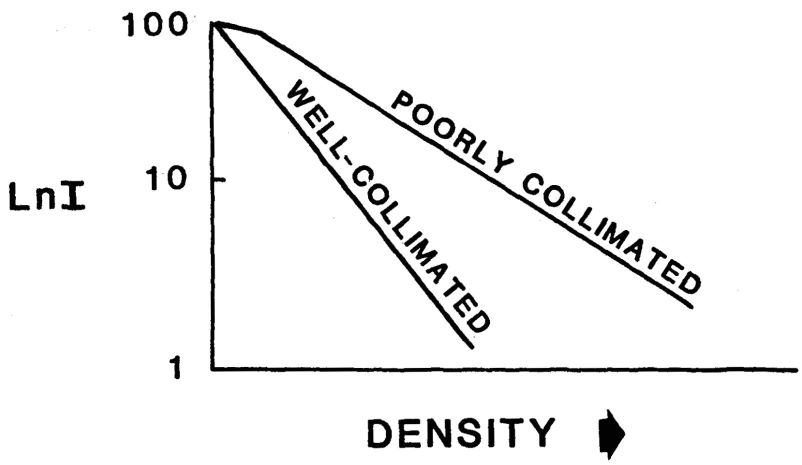
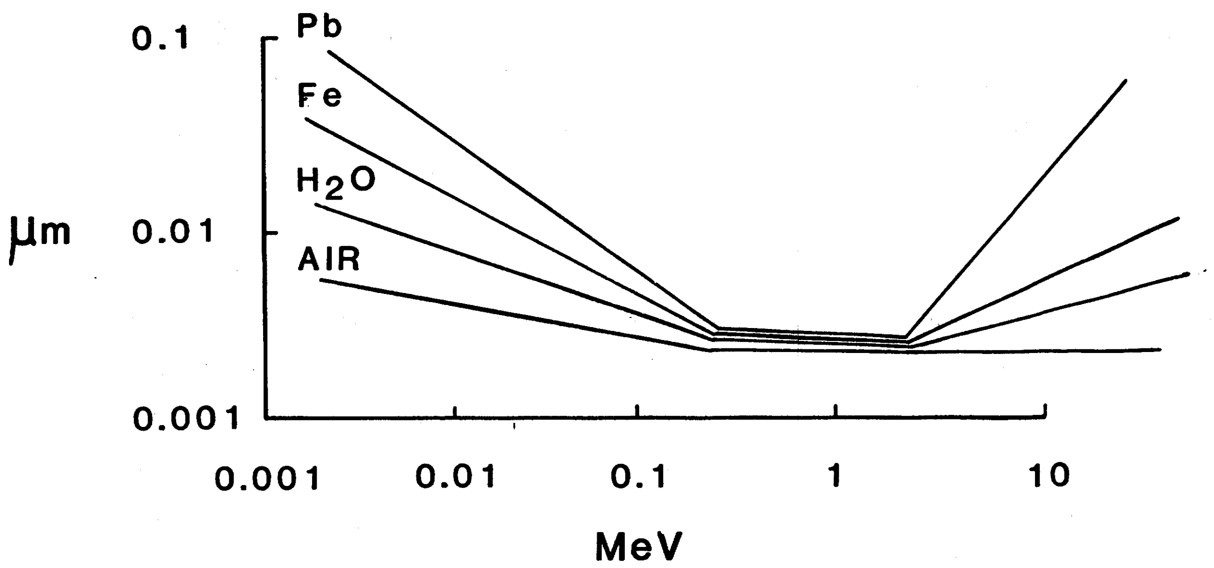


Fig. 18. Intensity of radiation detected for well-collimated and poorly collimated density gauges.



J = MeV x 1.6 x 10^{-13}

Fig. 19. Mass absorption coefficient (u_m) vs. energy for several materials.

Table 9. Relationship of count rates and counting time to nuclear gauge precision.

Count Rate	Counting Time	Total Count	$\pm$ 1 Std. Dev.	Percent of Total
(counts/minutes)	(minutes)	(counts)		(68% confidence)
10	1	10	3.3	33.
100	1	100	10.	10.
1,000	1	1,000	33.	3.3
10,000	1	10,000	100.	1.0
1,000	10	10,000	100.	1.0
100	100	10,000	100.	1.0

Precisions quoted (as standard deviations) by manufacturers (37) for direct transmission gauges range from 0.12 lb/ft^3 (1.9 kg/m^3) to 0.48 lb/ft^3 (7.7 kg/m^3) at a density of 120 lb/ft^3 (1922 kg/m^3). The lower (more precise) values are associated with longer count times, of the order of 4 minutes or longer.

To improve precision of a nuclear gauge measurement, high count rates and/or long counting times are necessary. High count rates are obtained by using large radiation sources or very sensitive radiation detectors.

Figure 20 is a diagram of a typical design for a direct transmission density gauge. Variations of this design, used by a large fraction of the State highway agencies (SHA), are commercially available from at least two manufacturers. These instruments were originally designed for measuring the density of soils. They contain a gamma ray source of about 10 millicuries (3.7×10^8 Bq) of Cs-137.

The source is encapsulated in stainless steel to prevent loss of the radioactive material. The source capsule is installed in the end of a metal rod that can be moved to positions below the surface of the soil or concrete. The positions are indexed with a locking position at 1- or 2-inch (25- or 50-mm) intervals along the rod. Maximum depth below the surface of the specimen is normally 12 inches (305 mm). Gamma radiation travels through the specimen to a radiation detector at the opposite end of the gauge body. The detector

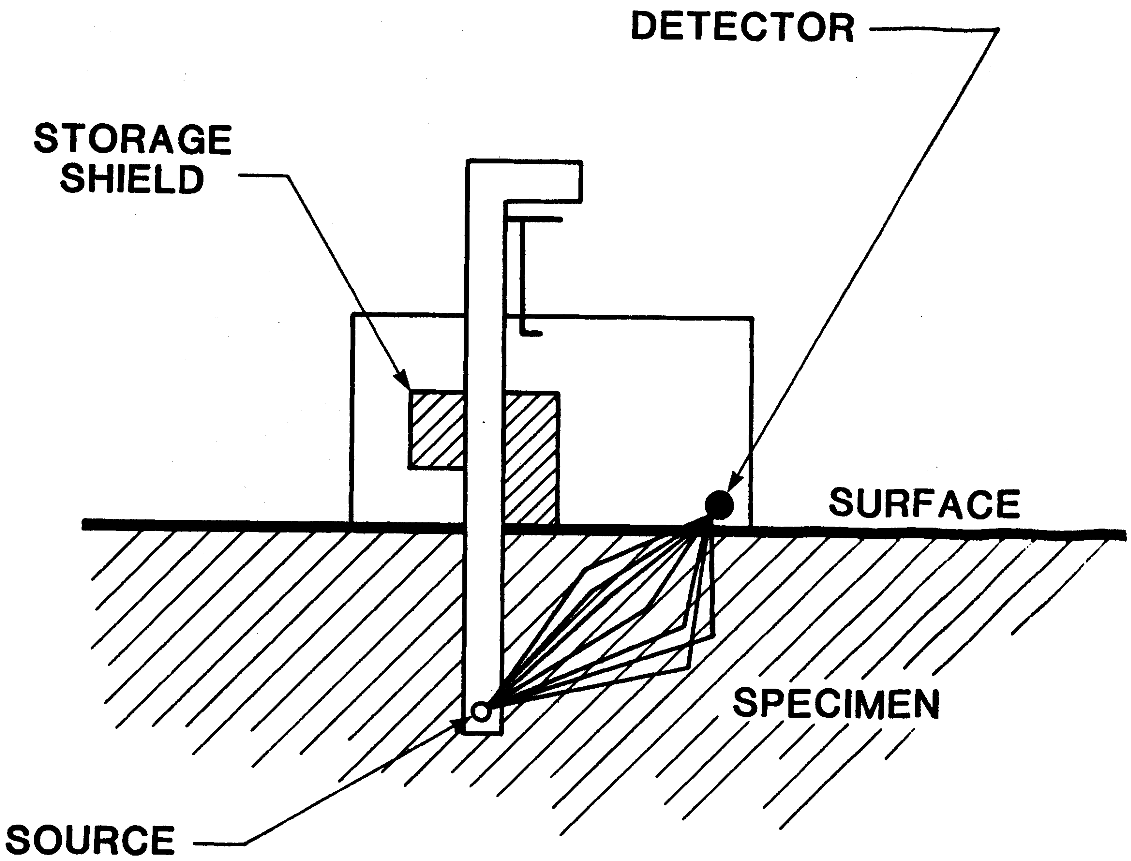


Fig. 20. Diagram of a typical direct transmission nuclear gauge.

is most often a Geiger-Müller (GM) detector. GM detectors are simple, rugged, stable detectors but do not offer energy discrimination. Since no energy discrimination and no collimation of source or detector exists, the radiation detected consists of the primary (full energy) and scattered radiation. Volume of the specimen contributing to the measurement roughly resembles a football with the ends at the source and detector positions. When not in use, the source is retracted into a shield inside the instrument.

b. Backscatter Gauges

A design for a typical backscatter gauge which is used for measurement of concrete density is diagrammed in figure 21. Both direct transmission and backscatter capabilities are contained within a single instrument, but the mode of operation is different. To operate as a backscatter gauge, the source is lowered from the shielded position to the bottom of the instrument just above the specimen surface. Only scattered radiation from the specimen is detected. The radiation detected bears a complex relationship to the density of the specimen. Figure 22 shows a typical response curve for a backscatter gauge. Actual response would be related to a number of factors including: source-specimen-detector geometry, energy of the radiation, elemental composition of the specimen, homogeneity of the specimen, and energy response of the detector. Geometric considerations are critical, with very small changes in geometry producing large changes in the gauge response. Composition and homogeneity of the specimen are probably the next most important considerations. Complexity of the situation can partly be indicated by an equation proposed for gamma backscatter gauges:

$$I = \frac{\mu_c I_o}{\mu_a + \mu_c} [1 - e^{-(\mu_a + \mu_c)t}] \qquad (7)$$

Note that there are two kinds of interaction coefficients, absorption (μ_a) and Compton scattering (μ_c). Each is actually a variable, changing with the energy of the radiation involved. Energy of the radiation decreases as it is scattered. The effect of each element in the specimen is different, especially as the energy of the radiation decreases, as was shown previously in figure 19.

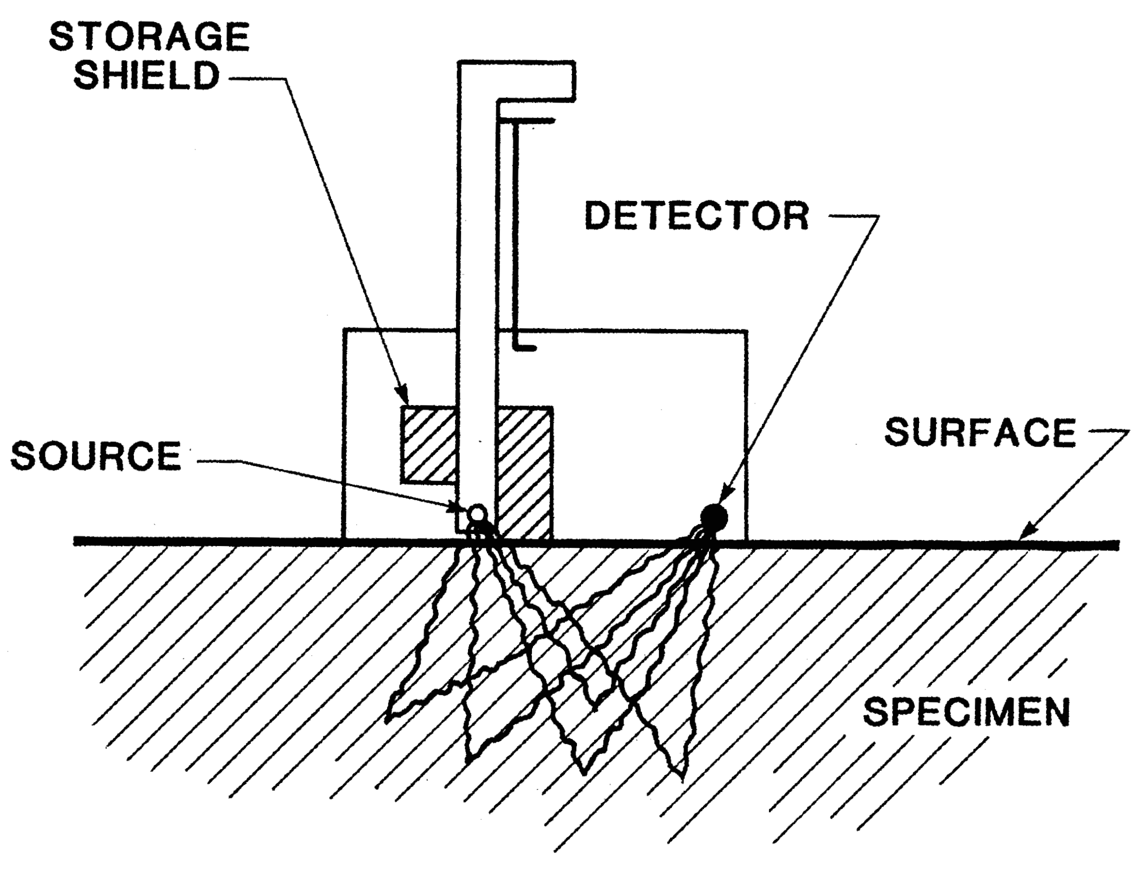
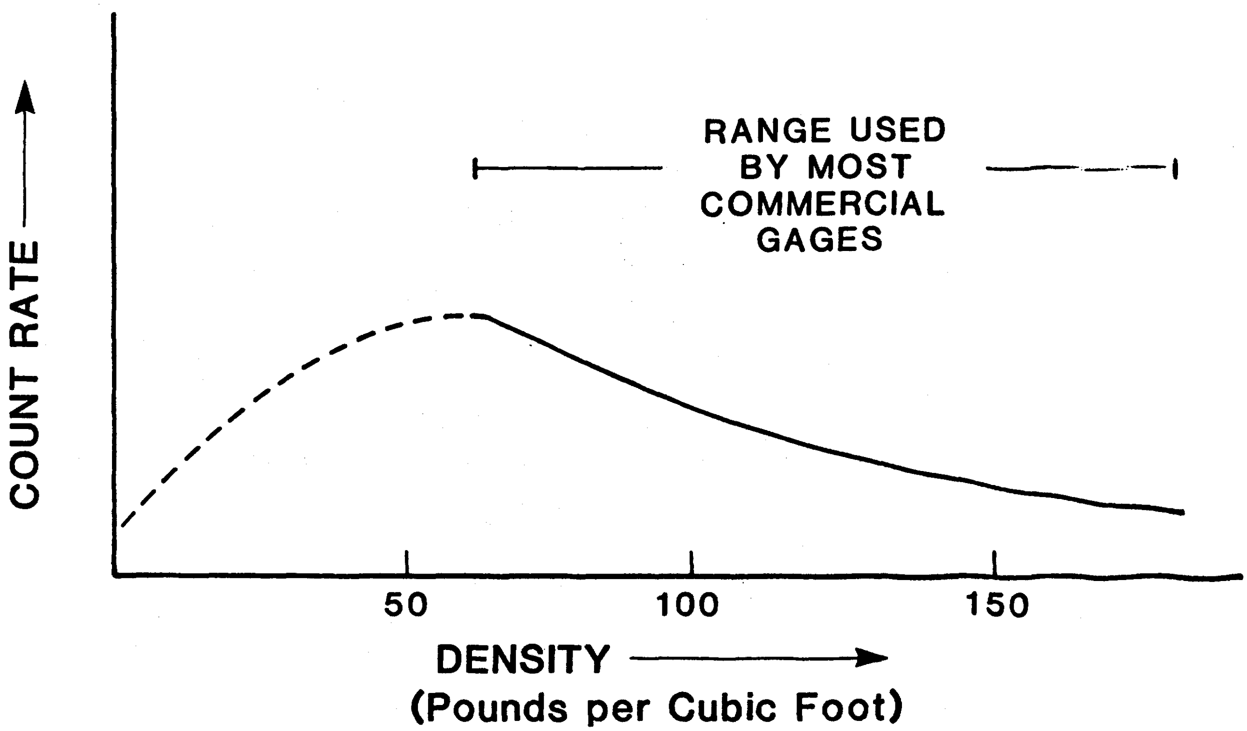


Fig. 21. Diagram of nuclear density gauge operating in backscatter mode.



$kg/m^3 = lb/ft^3 \times 16.018$

Fig. 22. Typical response curve for a backscatter gauge.

Due to these many potentially interfering factors, precision of backscatter gauges is typically less than that for direct transmission gauges. Quoted precisions range from 0.26 lb/ft^3 (4.15 kg/m^3) to 1.04 lb/ft^3 (16.6 kg/m^3) at a density of 120 lb/ft^3 (1922 kg/m^3). Backscatter gauges find their greatest application for density measurements on thin overlays of 2-inch (50 mm) thickness or less, on hardened materials, and on finished surfaces; in each of these cases direct transmission gauges would be either impractical or cosmetically unsatisfactory. On thicker sections, such as full depth pavements, direct transmission would be the preferred mode of operation.

c. Consolidation Monitoring Device (CMD)

Measurements using commercially available, static nuclear gauges suffer from a number of drawbacks, including inability to keep up with rapidly moving paving trains, and the need to make a large number of measurements in order to obtain representative values. A more promising approach is use of a continuous gauge. Such a device, termed a consolidation monitoring device (CMD), was developed under a previous contract to FHWA.[(38)]

While early models, which were modifications of oil well logging gauges, met with little success, this more recent instrument was designed specifically for measuring density (compaction) of concrete in highway construction. A diagram of the backscatter gauge portion of the instrument is shown in figure 23. The gauge utilizes a 500 millicurie (1.8×10^{10} Bq) Cs-137 source mounted in tandem with a NAI(Tℓ) scintillation detector and positioned on a slipform paver by a traversing mechanism. The source and detector are collimated so that only multiple scattered photons can reach the detector. The unit rolls along a horizontal guide beam as the paver traverses the pavement. The amount of gamma radiation scattered by the concrete back into the detector is proportional to the concrete density.

As variations in the air gap between the unit and the concrete surface were found to have a significant effect on readings obtained, an improved version allows for automatic compensation of the air gap via electronic capacitance sensing.[(39)] Field testing of the modified version of the instrument indicated that air gap compensation worked well over a 0.6- to 1.4-inch (15 to 35 mm) range.[(40)]

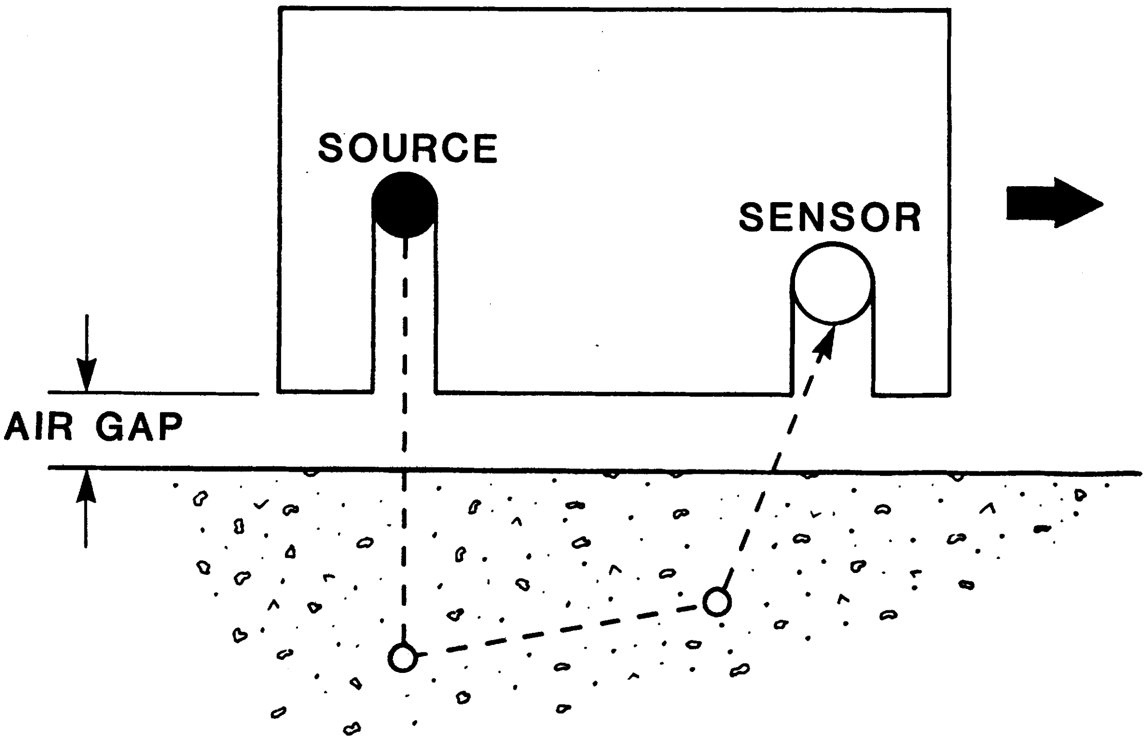


Fig. 23. Diagram of consolidation monitoring device (CMD).

Since the CMD is constantly moving over a new volume of the specimen, a different readout than for the static devices is necessary. The continuous gauge utilizes a ratemeter rather than a scaler and the information is most often presented as a strip chart recording. As such, a slightly different handling of the precision of the instrument measurements is necessary. This involves the setting of the ratemeter time constant.

The time constant is the time necessary for the ratemeter to reach 63 percent (1/e) of the applied signal. The time constant is actually a resistor-capacitor electronic circuit which averages the rate of electrical pulses received and transforms them into an electrical current. Selection of the time constant determines how fast the ratemeter will respond to an applied signal; i.e., a 5-second time constant selection will result in a "steady" reading by the ratemeter in less than 30 seconds.

As the time constant is shortened, the time in which the ratemeter reaches the new reading is shortened but the variation of that reading worsens. The mathematical relationships are:

$$V = V'' \left(1-\exp(-t/RC)\right) + V' \tag{8}$$

$$\sigma(\text{count rate}) = \sqrt{r/t} \tag{9}$$

where:

- t = the time of measurement
- V = the voltage at time t
- V'' = the increase in voltage at t=0
- V' = voltage at t=0
- R = the resistance in the RC network
- C = the capacitance in the RC network
- σ = the standard deviation (for count rate)
- r = the count rate

Figure 24 contains two graphs that compare the strip chart recording of the same count rate from a ratemeter-detector source system using different time constant settings. Note that the longer the time constant, the steadier is the readout, indicating better precision. This is just like information obtained from static gauges using a scaler, where the longer the time of measurement, the better is the precision of the measurement.

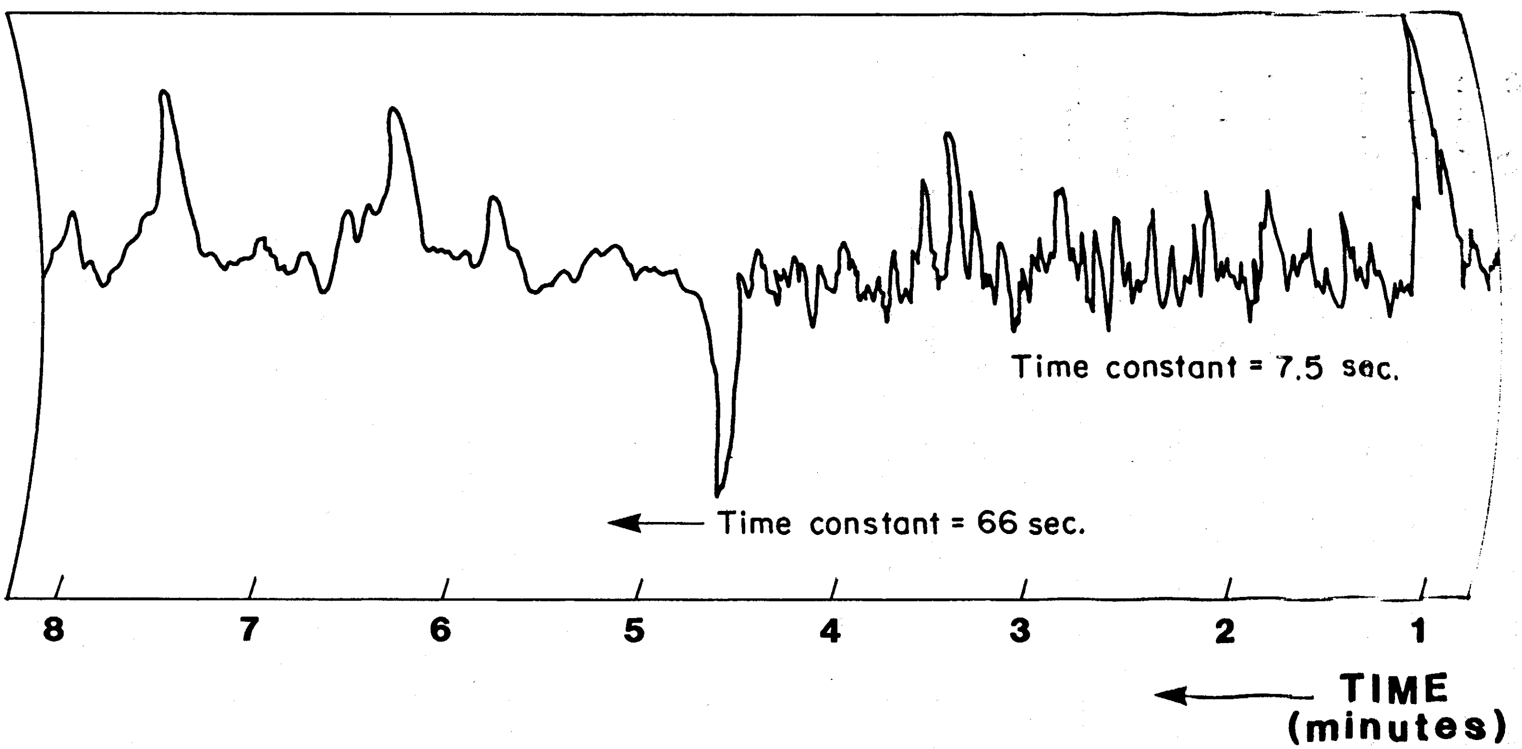


Fig. 24. Ratemeter recording from CMD.

Precision testing on the first generation CMD (uncompensated for air gap) indicated standard deviation of the difference between CMD readings and conventional densities (vibrated unit weight, weighed density, core density) to average from 1 to 1.5 lb/ft^3 (15 to 25 kg/m^3). More recent field studies[(41)] confirm the precision of the CMD to be within this range. While variability is therefore somewhat greater with the CMD than with static gauges, the ability to rapidly traverse large sections of pavement is seen as a major advantage for the continuous approach.

d. Twin Probe Technique

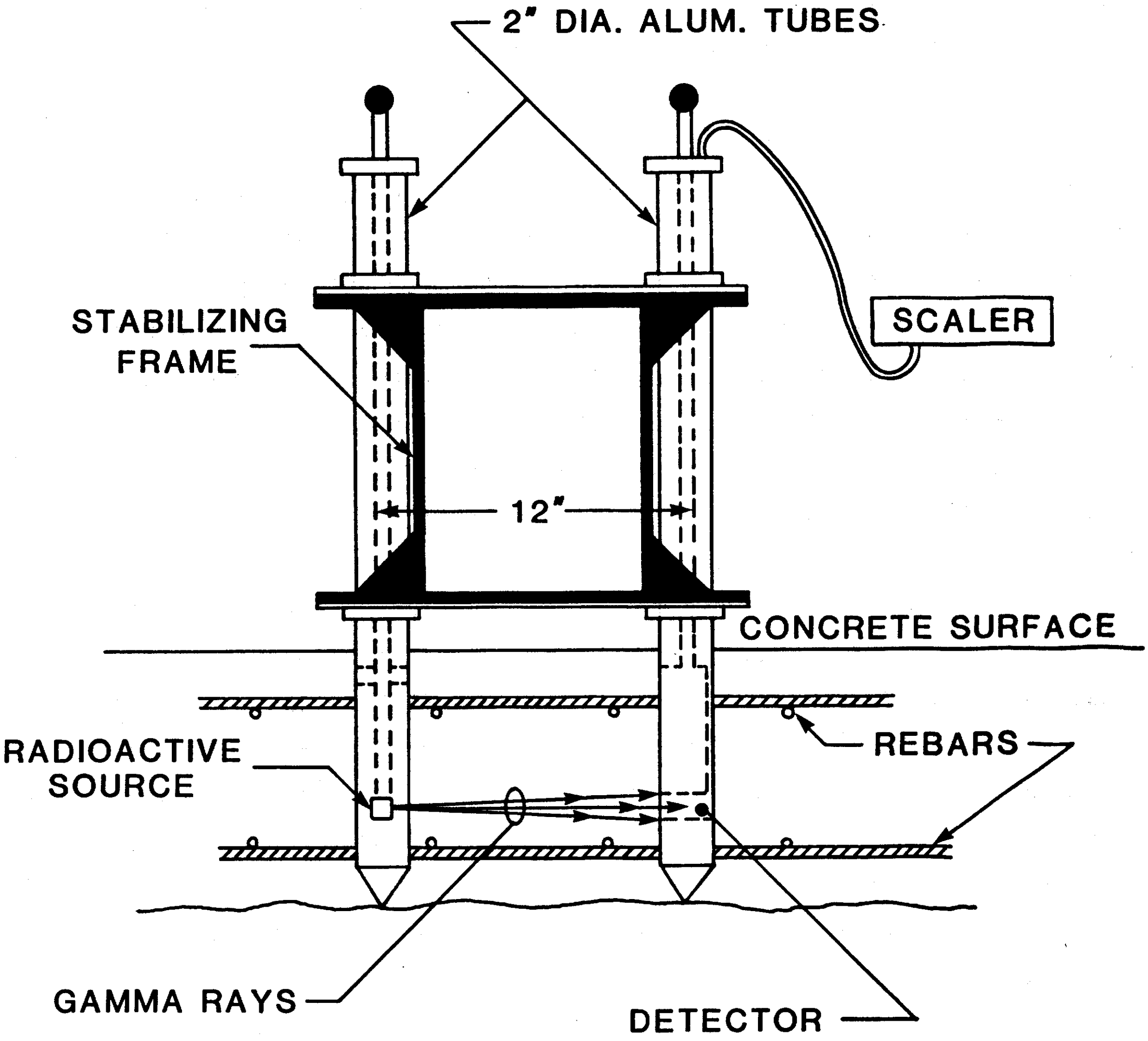
Both backscatter and direct transmission techniques suffer from the limitation that data on variability of density with depth is not well defined. In the case of backscatter gauges, approximately 80 to 85 percent of the response reflects the density of the top 2 inches (150 mm) of concrete. While direct transmission gauges will include a contribution from all concrete located between the source and detector, the density represents only an average value through the thickness being measured. In an attempt to overcome these limitations a commercial twin-probe source and detector system was modified by Iddings and Melancon for application to concrete.[(42)] A diagram is shown in figure 25. The source is Cs-137 encapsulated in stainless steel. No neutron source is included and only 5 mCi ($1.8x10^8$ Bq) of Cs-137 is used. The detector is a NaI(Tl) crystal that is 0.5-inches (12-mm) thick by 1.5 inches (38 mm) in diameter. The size of the crystal establishes the volume of the specimen measured, which is described by a pyramid with a base of 0.5 by 1.5 inches (12 by 38 mm) and with its apex at the source. Distance between the source and detector is set at 12 inches (305 mm). The counting system includes a pulse height analyzer which is set on the full energy gamma photon detection for the 0.66 MeV ($10x10^{-14}$ J) radiation gamma of Cs-137.

While this technique affords high sensitivity and good vertical localization of low-density areas, the device currently requires that the probes perforate the finished slab and thus is not amenable to continuous monitoring.

2. Current State Experiences with Nuclear Gauges

A large number of State agencies have utilized nuclear gauge technology in controlling consolidation of concrete. Some use nuclear density gauges on a

NUCLEAR TWO PROBE TECHNIQUE



mm = inches x 25.4

Fig. 25. Twin-probe nuclear density gauge.

routine basis, others have included their use in certain limited situations or on experimental projects. The information discussed in this section relates primarily to application of commercially available nuclear density gauges. Devices such as the CMD and two-probe depth gauge are considered experimental and are discussed in other sections of this report.

a. Previous Survey (1977)

In 1977 the Federal Highway Administration conducted a survey of State highway agencies with regards to the use of nuclear density gauges on concrete. Results of this survey were reported to TRB.[(43)] At that time, 29 States had used or were using nuclear gauges to varying extents on portland cement concrete. Of these States, 11 used the gauges on a routine basis for consolidation monitoring, primarily on low slump, "dense" concrete (LSDC) bridge deck overlays. Use on full depth deck and pavement placements was limited. Use of gauges on overlay applications was about equally divided, in terms of number of States, between backscatter and transmission gauges.

The FHWA summary included concise discussions of various parameters influencing gauge performance. These included depth effects, chemical composition, and steel location. Such effects were included in this study as part of the analysis of gauge performance, and are discussed in detail in section 4, along with information on gauge accuracy and calibration. Efforts to include use of gauges in specifications were also discussed in the 1977 survey; these are updated in this report. Finally, useful information on radiation safety was provided in the FHWA report. Little has changed in this regard since 1977, with standard radiation safety procedures being followed by personnel using the gauges.

b. 1985 Survey

As part of the present research program, a second survey of nuclear gauge users was undertaken. States responding to the 1977 survey were contacted, and individuals responsible for gauge activities were queried via letter and telephone. Thirty-two States were included in the 1985 survey. Of these, 7 reported no use of nuclear gauges. Three others were believed to be using, or had used, nuclear gauges, but did not supply any additional information. This left a total of 22 States which supplied information on gauge applications.

While use of nuclear gauges on LSDC overlays is popular, use of such techniques apparently is not greatly increasing at the present time. Twelve (as opposed to 11 States in 1977) are routinely using the gauges on LSDC overlays at present. Many of those States reported as experimenting with or planning use of the gauges in 1977 apparently did not adopt use of nuclear consolidation monitoring as a routine procedure.

c. Highlights of State Experiences

Pertinent information obtained from the current survey is summarized in table 10. A number of key items deserve comparison among the various States utilizing nuclear density gauges.

(1) <u>Gauge Mode</u>. Of the States surveyed, ten use direct transmission and six use backscatter mode. Three States use both modes, the choice of mode primarily dependent on the concrete application. Where thicker sections such as pavements are being tested, the transmission mode is preferred. Two of the States surveyed used other techniques or did not specify the mode utilized.

(2) <u>Construction Applications</u>. The predominant application of nuclear gauges is still in monitoring consolidation of LSDC overlays. Seven States reported use on other types of structures, these being bridge deck pours and pavements. Of the six States having experience with their use on pavements, three have used the gauges only once on experimental projects. Of the remaining three States (Colorado, Kansas, and West Virginia), Colorado has not used the gauges on pavements for the past two years, and Kansas uses them only occasionally on special projects. Use in West Virginia is apparently more widespread but further information was not supplied.

(3) <u>Precision and Accuracy</u>. Controlled studies on precision and accuracy of static nuclear density gauges as applied to PCC are lacking. "Accuracy" in these applications is difficult to assess. The volume sensed by the nuclear gauges is ill defined, making it very difficult to determine density on exactly the same concrete measured by the gauge. There is some disturbance of concrete with transmission and two-probe devices. The only absolute reference method, the taking of cores from the concrete, measures the density of

Table 10. Summary of State highway agency experience with nuclear density gauges.

STATE	GAGE MODE	TYPE CONSTRUCTION	TEST PROCEDURE	PRECISION AND ACCURACY	CHECKED VS. CORES	INTERFERENCES	RUGGEDNESS	GENERAL ASSESSMENT AND USE
Alabama	None Used							
California	Troxler 2400 Campbell Pacific MC-1 Direct Transmission	Thin-bonded overlays	CALTRANS	± 2.0 lb/ft^3 vs. cores	Yes	Steel reinforcement	"Rugged and Practical"	Not routinely used. Only occasional use on overlays.
Colorado	Troxler 2401, 3401 Campbell MC-1A Direct Transmission or Backscatter	Pavements Bridge decks	CP-82	Backscatter -1.5 lb/ft^3 vs. rodded U.W. Transmission +0.4 lb/ft^3 vs. rodded U.W.	Yes Av. error 0.5 pcf Range +2 to -1.8 pcf	Steel if 2" deep	Performs well in field	Very good performance. Not used in past 2 yrs on pavements. Considering eliminating from bridge decks.
Georgia	Troxler 2401-Direct Transmission	Paving	N/A Plastic Wrap Below Gauge	"Gave reasonable results"	No	Not studied	N/A	Used only once
Iowa	Troxler 2400, 3401 Campbell Pacific MC-1 Direct Transmission	LSDC overlays	IDOT Memo 358	N/A	N/A	Embedded steel	Generally rugged and dependable	Considered satisfactory in achieving desired results
Idaho	None Used							
Illinois	Troxler-Backscatter	LSDC	IL DOT	"Feels they are accurate to ± 1.5 lb/ft^3"	"Erratic Results" N/A	Steel in path	Tolerates normal abuse	"Fast, practical, reliable" Indicates ± air content
Indiana	Troxler-Backscatter	LSDC	Indiana	"Excellent": use open-box calib.	No	None: If steel 2.5"	N/A	Good results when calib. on job using Indiana procedure
Kansas	Troxler 2401, 3401, 3411 Direct Transmission	Overlays Pavements	KSDOT KT-38-84	Corr. Factors. +3.4 lb/ft^3 w/mesh - 2.9 lb/ft^3 w/o mesh	N/A	Avoid steel	Good. Periodic cleaning advised	Useful tool. Use is optional
Kentucky	Seaman C-75, C-100, C-200 Troxler 3411-B Backscatter	LSDC	Not Specified	"Suspect" Tolerance ± 1.5 lb/ft^3 vs. calibration blocks	On asphalt only	N/A	Tolerates abuse, some electronic failures	"Fast and fairly accurate"
Michigan	Troxler 2401 Troxler 3411B Direct Transmission	LSDC	Michigan	N/A	No	None if 2" from steel	N/A	Workable. Not used at present
Minnesota	None used							
Mississippi	Troxler-Direct Transmission	Paving	Not specified 4 tests per location	Std Dev 1.1 lb/ft^3 Av. Error ± 2.5 lb/ft^3 vs. rodded U.W.	Yes-See NEEP study	N/A	N/A	Only used on NEEP Study. Not in general use
Missouri	Troxler 3401B - Direct Transmission	LSDC	MHTD T36-8-84	N/A	No	Steel: In backscatter mode.	Satisfactory. Vibration may disrupt wiring.	Satisfactory and Practical
Montana	Direct Transmission	LSDC LMC	Colo CP-82	Limited data. Results generally 100-104% of rodded U.W.	No	Avoid steel	N/A	Not very useful.

Table 10. Summary of State highway agency experience with nuclear density gauges (continued).

STATE	GAGE MODE	TYPE CONSTRUCTION	TEST PROCEDURE	PRECISION AND ACCURACY	CHECKED VS. CORES	INTERFERENCES	RUGGEDNESS	GENERAL ASSESSMENT AND USE
Nebraska	Troxler 2400, 3400 Campbell MC-1, BRC Direct Transmission	LSDC	Not specified	"Appears to be accurate"	N/A	None	Well-built & rugged enough	Satisfactory.
New Hampshire	Not specified	LMC	For moisture content only	Reads 0.7% low.	No	N/A	N/A	Only used once for moisture content.
New Jersey	None Used							
New Mexico	No Information Supplied							
New York	Troxler 3411B Direct Transmission	LSDC	NYSDOT MM9.5	Nuclear & unit wt. agree to within "acceptable limits"	No	Away from large structures. 30' from another gage. Not on track beds. Avoid steel.	Problems with electronics	Very practical. Quick, easy, accurate
Nevada	No Information Supplied							
North Carolina	Backscatter	Bridge deck overlays	NC DOT Procedure		No			
Ohio	None Used							
Oklahoma	Troxler Campbell Pacific	LSDC	Not Specified	N/A	N/A	N/A	N/A	N/A
Oregon	Troxler - 3401 Campbell Pacific MC-1 Backscatter	LSDC Bridge Decks	Not Specified	N/A	No	Reinforcing Steel	Rugged	Satisfactory if used by trained operator. Very practical.
Pennsylvania	Troxler 3400, 2400 Campbell MC-1 Backscatter	LSDC	PENNDOT	Overall ± 5 pcf vs. rodded U.W. Often not ±2.5 lb/ft^3 rodded U.W.	No	Rebar: no effect Some error due to compaction by gage	"Adequate" Some electronic failures	Very satisfactory Limited since 1979
Tennessee	None Used							
Utah	Lane-Wells Road-logger	Pavement	Not specified	Ave. error 3.5 lb/ft^3 over unit wt.	Yes Limited data.	N/A	N/A	Used only once on experimental project
Virginia (VHTRC)	Troxler 3411 Backscatter & Direct-Transmission	LSDC CRCP	See report	CV 0.5% at single position 0.9-1.8% in field Corr. Factors 0.3-1.5 lb/ft^3 Backscatter. 6.0 lb/ft^3 Direct transmission.	Petrographic Exam.	N/A	N/A	Fails to determine voids which are potentially deleterious
W. Virginia	Troxler 3411B	Pavement Decks, Shoulders	AASHTO T271		No	Steel. Sample inhomogeneity	N/A	N/A
Wisconsin	No Information Supplied							
Wyoming	None used.							

hardened concrete, which may be different from the density in the fresh state, depending on such factors as extent and type of curing, shrinkage, and moisture content at time of test.

For these reasons, gauge performance is better evaluated by its precision, taken here as the reproducibility of results on any given test site. It should be recognized that inherent variability in the concrete itself will make the measurement of precision also somewhat open to interpretation. Other measurements of accuracy are the so-called "correction factors," that is, the difference between results as measured by the gauge and density of concrete from the same mix compacted using a standard technique. In some cases, there may be as much (or more) variance in the techniques used to consolidate the "standard" sample as there is in measurement of density by the nuclear methods. This is supported somewhat in the wide variation in correction factors reported in the current survey. Factors as low as 0.4 lb/ft^3 (6.4 kg/m^3) and as high as 5 lb/ft^3 (80 kg/m^3) have been reported. The most often cited "accuracies" ranged from 1 to 2 lb/ft^3 (16 to 32 kg/m^3); however, solid data to support these figures were not available.

(4) Interferences. The primary interference in nuclear density gauge testing is the presence of reinforcing steel or other metallic elements within the field of vision of the gauge. Most SHA's surveyed apparently recognize this limitation and noted such in their responses. The minimum distance between the gauge and embedded steel was seen as about 2 to 3 inches (50 to 75 mm) in most instances.

(5) Ruggedness Assessment. While most States surveyed felt that nuclear gauges were sufficiently rugged for field use, there were a few problems noted, especially with regards to electrical connections within the gauges. Kentucky noted that some gauges failed electronically prior to field service, indicating some preexisting malfunction either at manufacture or during shipping. Vibrations during transit can disrupt wiring, as noted by Missouri personnel. Electrical repairs were also required on gauges used by New York and Pennsylvania.

(6) General Assessment. In general, most SHA's surveyed found the nuclear density gauges to be a useful and practical tool for measurement of density of PCC. Performance was considered satisfactory, although in many cases use was limited so conclusions were drawn from experiences on only a few projects. The rapidity of the measurement was cited as a major advantage, as was the ability of the gauge to detect other changes in concrete mixtures, such as large variations in air content.

In spite of these positive evaluations, as mentioned previously, use of the gauges has not increased appreciably since the 1977 survey. No attempt has been made to systematically document the reasons for this lack of general acceptance; however, hindrances might include: the belief that consolidation generally is satisfactory and does not need to be monitored, increased costs for inspection and for training and certification of operators, licensing and safety requirements associated with nuclear technology, early unsatisfactory experiences, and increasing use of workability aids, such as latexes and "superplasticizers," which make full consolidation easier to achieve. While such concerns certainly may be justified, the costs associated with even a few failures due to lack of consolidation are great, and an ongoing consolidation monitoring program using nuclear density gauges offers a relatively low-cost alternative for avoidance of such problems.

d. Current Test Procedures

In conjunction with the survey of State nuclear gauge experiences, information on current test procedures was solicited. Eleven States supplied copies of their current test procedures. While these procedures are similar in many respects, there are important differences which merit some discussion. Also included in this comparison is the new (1985) ASTM Designation: C 1040 (Standard Test Methods for Density of Unhardened and Hardened Concrete In Place by Nuclear Methods). A summary of pertinent information relating to these test procedures is given in table 11.

(1) Reference Calibration. "Reference calibration" refers to the procedure used to originally establish the relationship between count rate and density. The majority of States use the information supplied by the manufacturer in the gauge certification information. Two States (California and Iowa) perform

Table 11. Summary of State nuclear density test procedures.

State	Reference Calibration[1]			Standardization Procedure[2]			Field Calibration Vessel				Field Calibration Procedure[4]			
	Gage Mode	Calib Std	Procedure	Ref Std	No Readings	Tolerance[3] Check	Frequency	Material	Size	Consol	No. Readings	Geometry	Time Period	Frequency
California	Transmission	High Silica High Calcium	Four 1-min Readings Plot Curve	---	---	---	---	Aluminum	24"x24"x4"	Int. Vib.	1	Near Center	---	Daily
Colorado	Pavement-Transmission Deck - Transmission Backscatter	---	---	"Portable"	One 4-min	±1.96 Av CR	Daily	---	---	---	---	---	---	---
Illinois	Backscatter	---	---	Supplied by Mfr.	One 4-min	±1% of Av of previous four	Daily, midway, and at changes	---	27"x21"x overlay thickness	Vib.	1	Center	4 min.	1 per lane
Indiana	Backscatter	---	---	---	---	---	---	Plywood	16"x20"x1-5/8- Open bottom	Marshall Hammer 10 lb-8 in. drop, 2 blows	4	180° reversal	1 min ea.	---
Iowa	Transmission	Blocks of granite and concrete of known density	Five 1-min readings	Supplied by Mfr.	One 4-min	±1% of last count	Twice daily	High density concrete	20"x20"x7"	---	5	---	1 min ea.	Not < every 3 months
Kansas	Transmission	---	---	---	---	---	---	---	18"x18"x10"	Vib. two lifts	3	12 ea. angle 0°, 90°, 180°	1 min ea.	---
Michigan	Transmission	---	---	Supplied by Mfr.	One 4-min	---	Daily	---	20"x14"x14"	Spade. Drop thru 3-in. 4-6 times	1	Center	1 min	Daily
Missouri	Transmission	---	---	Supplied by Mfr.	---	---	---	Steel	24"x24"x4"	Spade. Drop thru 3-in. 4-6 times	4	Center	1 min ea.	---
New York	Transmission	---	20 1-min readings	Supplied by Mfr.	Four 1-min	1% shift	3 times daily	---	---	---	---	---	---	---
North Carolina	Backscatter	---	20 1-min	Supplied by Mfr.	1	---	Daily	---	27"x21"x4"	Spade. Drop thru 3-in. 4-6 times	1	Center	1 min.	Daily
Pennsylvania	Backscatter	---	---	Supplied	---	---	---	---	18"x12"x4"	Vib. until box density = vibrated density of U.W. bucket	15	Center-5@ 20°,5@180°	---	---
ASTM C 1040	Transmission Backscatter	Materials Uniform Density	---	Supplied by Mfr.	5	Use Eqn. (2) in method	When composition changes		24"x24"x4" 18"x18"x6"	Use ASTM C 138 or other method	12	Center-4@ 90°, 180°, 270°	minimum	Daily

Notes:
1. Reference Calibration Refers to calibration used to establish relationship between density and count.
2. Standardization Procedure refers to determination of standard count, usually on a daily basis.
3. Test is normally repeated if results fall outside of stated tolerance limits.
4. Field Calibration refers to procedures used to develop] correction factor for concrete job mix.

Table 11. Summary of State nuclear density test procedures (continued).

Measurement							
				Tolerances			
State	Depth	Time Period	Frequency	Edges	Steel		Specification Notes
California	2-in	---	Daily	---	---	Not <98% Rodded	
Colorado	Pavement - 2 in less than thickness - Deck 1-in if transmission	1 min	---	Not <12 in	Not <2 in	Class P - Not <96% Max Theo. Field Density Class AX - Not <94%	1. Specifications currently under revision 2. Correct for air content
Illinois	---	1 min	Every 10 ft	---	Not <3 in	Not < Weighed density	
Indiana	---	---	Three tests per set Not <1 set per 50 ft	---	Not <1-1/2 in	Not <98% Rodded	
Iowa	2 in	1 min	---	---	Avoid steel	Not <98% of Standard	1. Correct for air content
Kansas	8 in	1 min-Av 3 counts	In vib path & between	Not <1 ft	Not <2 ft dowel basket	Not <98% of Rodded	
Michigan	1-1/2 in	1 min	3 points 1st 10 ft 3 points every 100 ft	---	---	Not <98% Max Density (From Calib. Box)	1. Not used since 1978
Missouri	1-1-1/2 in less than thickness	1 min	---	---	---	---	
New York	2-in	1 min	1 reading per 12 ft lane per 20-30 linear ft	---	Avoid steel	Not <98% Theo. Unit Wt.	
North	---	1 min.	---	---	---	---	
Pennsylivania	----	10 readings	---	---	---	---	1. Little use since 1979
ASTM C 1040	Transmission 2 - 12 in	1 min	---	Not <9 in	Steel not in meas. volume	---	

their own calibrations using materials of known composition. If these calibration materials are close to concrete in their composition, this approach may have advantages in that field correction for chemical effects may be reduced.

(2) <u>Standardization Procedure</u>. "Standardization procedure" refers to the method of determining the standard count of the gauge that is, the number of counts for a fixed interval on a standard material of uniform composition. Normally, the reference standard used for this procedure is supplied by the manufacturer. For contemporary gauges such materials as laminated polyethylene/magnesium are commonly used.

A single reading of up to four minutes duration is used by most SHA's for the standard count, although the ASTM procedure specifies five readings and New York uses four 1-minute readings.

As the purpose of the standard count is to maintain a check on gauge stability, many SHA perform a reference check on a daily basis. The ASTM method specifies that the difference between two successive daily counts on the reference standard should be less than $1.96\sqrt{N_0}$, where N_0 is the previously established count. Colorado uses this limit; however, other States have set simpler limits, usually in the range of 1 percent of the previous value. Depending on the absolute value of the count, this may be more or less stringent than the ASTM recommendations.

(3) <u>Field Calibration Vessel</u>. In order to compensate for varying compositions of field materials, many gauge users choose to perform a field calibration using the same concrete employed on the actual job. To accomplish this, the concrete must be consolidated into a vessel from which an independent measure of density can be made. Construction and procedures vary widely in this regard. Materials used to fabricate the vessel have included wood, steel, and aluminum. A variety of sizes are in use as can be seen from table 11. In an attempt to compensate for substrate affects, Indiana utilizes an open-bottom box, so that the density contribution of the concrete deck can be included in the calibration.

A variety of procedures are used to consolidate the concrete. These include internal vibration, use of a Marshall hammer, and spading and dropping. Both the total amount of consolidation achieved by these methods and the reproducibility of the method each time a calibration is run will have an impact on results obtained through subsequent readings using the nuclear gauge.

(4) Field Calibration Procedure. There is also quite a wide variance in procedures used for determination of nuclear gauge readings on the calibration vessel. Some States take only a single reading, others obtain an average of five readings. While a few States rotate the gauge through a fixed angle after each reading so as to average out any inhomogeneities in the concrete, this is limited generally to the central portion of the vessel. Any fluctuations in composition at points removed from the center will not be reflected in the gauge readings but will be included in the weighed density. Frequency of calibration as related to the progress of construction also varies among States. While daily calibration is generally recommended, Iowa limits this to not less than every 3 months, and Illinois calibrates once on each lane being measured.

(5) Measurement Procedure. Important variables associated with the actual measurement of density include the depth of measurement, time period, and frequency of test. Depth of measurement primarily applies to readings obtained in transmission mode. On bridge deck overlays, such measurements are limited by depth of placement of overlay. A depth of 2 inches (50 mm) is the deepest transmission measurement taken for this application (Colorado, Iowa, and New York). On full depth pavement, the approach is to include as much of the pavement thickness as possible in the measurement.

Most of the SHA (as well as ASTM) utilize a 1-min time period for the test count. Kansas averages three of these 1-min counts. Pennsylvania averages 10 counts.

There is wide diversity in recommended frequency of testing. Five of the 11 States do not specify any particular frequency of measurement. Kansas specifies location but not frequency. The other SHA range from daily readings to readings every 10 ft (3 m). The more frequent readings would be more practical on smaller jobs, such as bridge deck overlay placement, but might become unmanageable on full-width paving operations.

(6) Tolerances. Tolerances are normally set on minimum distances between the nuclear gauge and construction edges or steel components. Those States specifying tolerances on distance from edges recommend that the nuclear gauge be positioned not less than 12 inches (300 mm) from an edge (ASTM recommends not less than 9 inches (230 mm)). There is somewhat less agreement with respect to minimum distance from steel. Some States simply recommend avoiding steel; for others minimum distances range from 1 1/2 inches (38 mm) from reinforcing steel to 2 ft (0.6 m) in the case of dowel baskets.

e. Specifications

The objective of nuclear gauge testing is to allow rapid, real-time control of consolidation so as to enable inspectors to make judgments as to acceptance or rejection of concrete inplace. Most SHA's using nuclear gauges in acceptance have adopted a lower limit of 98 percent of some "standard" density (normally rodded unit weight). That is, if the reading obtained with the nuclear gauge is less than 98 percent of the density obtained by consolidation of concrete using a standard procedure, then remedial action must be taken. This action may involve revibration or removal of concrete. It should be noted that the standard procedure itself may not completely consolidate the concrete, and therefore values greater than 98 percent of standard may not necessarily represent full consolidation.

3. Application of Nuclear Density Gauges to Highway Construction

While a number of agencies have carried out evaluations of both static and continuous nuclear density devices, their use to date has been limited either to specific types of construction (such as bridge deck overlays) or to experimental projects. Consolidation of freshly placed concrete as a basis for payment has not generally been included in end-result specifications. This may be due to an unfamiliarity of many highway practitioners with use and applicability of nuclear gauges to full-scale highway construction. Further work needs to be done in terms of developing such information so that consolidation monitoring can be implemented into statistically-based quality assurance programs. In this section gauge characteristics pertinent to various phases of highway construction will be summarized, and the advantages and disadvantages of each gauge type discussed.

a. Precision of Nuclear Density Gauges

As mentioned in discussion of results of the 1985 survey (section 4), precision data with respect to application of nuclear gauges to PCC are somewhat difficult to define. In routine testing, determinations of density are made at widely spaced distances, so that most likely there will be some difference in actual density of the concrete between test positions. Any precision statistics derived from a series of such measurements will reflect both the inherent variance of the gauge plus the actual variance of concrete density. Examination of reported results, however, does allow one to establish a rough upper bound for gauge precision, assuming that the process is fairly well controlled and that the concrete density does not undergo unduly large fluctuations. Examples of such data, taken from a number of recent projects in a variety of States, are shown in table 12.

Standard deviations of the nuclear gauge readings for these field projects range from 1.2 to 2.8 lb/ft^3 (19 to 45 kg/m^3), with the majority less than 2.2 lb/ft^3 (35 kg/m^3). Considering that these values include variations in the concrete as well as variations in gauge response itself, these data indicate that precision may be satisfactory for most intended purposes in field applications. Standard deviation for rodded density in most instances is somewhat less than for nuclear density.

In a somewhat more controlled field study, research carried out in Mississippi under the NEEP program allowed an estimate to be made of the precision of direct transmission nuclear gauges.[44] In this study, readings were taken in groups of four, with each measurement spaced approximately 3 ft (1 m) apart. Standard deviations for 48 sets of such measurements ranged from 0.3 to 1.7 lb/ft^3 (5 to 27 kg/m^3) with an average standard deviation of 1.1 lb/ft^3 (18 kg/m^3).

Additional precision data derived from research-oriented field studies are reported by Scholer and Schumm.[45] In this study direct transmission density readings were taken at various locations on a slipform concrete paving project. Results are summarized in table 13.

Table 12. Precision data for various projects using nuclear density gauges.

State	Project	No. Readings	Rodded Density Av. (lb/ft^3)(1)	Rodded Density Std. Dev. (lb/ft^3)(1)	Nuclear Density Av. (lb/ft^3)(1)	Nuclear Density Std. Dev. (lb/ft^3)(1)
No. Carolina	8.1744402	189	146.6	1.7	148.5	2.2
"	8.1625210	207	145.1	0.6	148.0	1.2
Colorado	I76-1(53)24	57	146.7	1.9	142.8	1.8
New York[2/]	250363	84	142.4	-	141.4	1.7
"	250363	67	142.4	-	145.0	2.2
"	250548	33	142.6	-	144.0	2.1
"	250548	46	142.6	-	144.8	1.9
"	250551	71	143.2	-	144.0	2.8
"	250551	40	143.1	-	142.5	2.1
"	250551	147	143.2	-	145.9	1.7
W. Virginia	APD-483(31)	59	141.0	1.7	139.2	2.5
Illinois[(3)]	Riverside Bridge	52	142.5	-	141.3	2.5
					Av.	2.0

1. kg/m^3 = lb/ft^{3}x16.018.
2. "Rodded" values represent maximum theoretical density.
3. Rodded density determined at start of project only.

Table 13. Summary of tests on I-64 slipform project.[45]

Location	Average (lb/ft^3)(1)	Standard Deviation (lb/ft^3)(1)
<u>All Tests</u>		
Above reinforcement	146.4	1.8
Below reinforcement	145.6	1.7
<u>At Contraction Joints</u>		
Above reinforcement	145.5	1.8
Below reinforcement	145.7	1.7
<u>Not at Contraction Joints</u>		
Above reinforcement	145.1	1.5
Below reinforcement	145.5	1.6
All Tests	146.0	1.8

(1) kg/m^3 = lb/ft^{3}x16.018.

Results agree favorably with those of the previously cited studies with standard deviation ranging from 1 to 2 lb/ft^3 (16 to 32 kg/m^3).

One method of extracting gauge variability from concrete variability in field studies is to examine variance of standard counts determined over the course of a project. One such set of data is available from information supplied by the New York State Department of Transportation for field work in 1983 and 1984 and by the Colorado Division of Highways in 1980 through 1985. The variation in standard count can be expressed as an equivalent density as shown in tables 14 and 15.

The gradual decrease in the standard count with time represents decay of the Cs-137 source. This, of course, is the reason for calculating concrete density from a ratio of the standard count and the specimen count, in order to cancel out isotopic decay and instrument component aging. This slow aging, however, is insignificant and is not reflected in daily estimates of gauge precision using this approach, which average 0.2 lb/ft^3 (3 kg/m^3) for the first set of data and 0.6 lb/ft^3 (10 kg/m^3) for the second.

While results of routine field measurements afford a general appreciation for the range of gauge precision to be expected, more reliable information can be obtained from repeated measurements on the same sample. The amount of such

Table 14. Standard count variations recorded by NYDOT and equivalent calculated changes in concrete density (Gauge #10261, NYDOT)

Date	Standard Count Start	Standard Count End	% Change	Equivalent Density Change (lb/ft^3)(1)
5/24/84	3421	3412	0.26	0.20
5/26/84	3415	3434	0.55	0.38
5/31/84	3420	3423	0.09	0.06
7/09/84	3384	3394	0.30	0.20
7/11/84	3382	3389	0.21	0.14
7/13/84	3379	3389	0.30	0.20
			Average	0.20

---

[1]$kg/m^3 = lb/ft^3 \times 16.018$.

Table 15. Variation in standard count recorded by Colorado Department of Highways and equivalent calculated change in concrete density

Date	Standard Count	Variation	Equivalent Density Change (lb/ft^3)(1)
7/29/80	6151	46	0.6
7/30/80	6136	31	0.4
7/31/80	6156	51	0.7
8/01/80	6128	23	0.3
8/04/80	6150	45	0.6
8/05/80	6165	61	0.8
9/18/80	6129	24	0.3
9/22/80	6064	-56	0.6
9/23/80	6137	32	0.4
9/25/80	6186	81	1.1
9/26/80	6148	43	0.6
		Average	0.6

---

[1]$kg/m^3 = lb/ft^3 \times 16.018$.

information available, however, is limited. Work reported by Ozyildirim gave a coefficient of variation of 0.5 percent for sets of 15 readings without moving the gauge.[46] This represents a standard deviation of approximately 0.75 lb/ft^3 (12 kg/m^3). Further precision data derived from multiple measurements are presented in table 16 for data derived during initial development of the consolidation monitoring device.[38] Standard deviations for these data are quite low, averaging 0.4 lb/ft^3 (6 kg/m^3). Additional estimates of precision based on multiple readings were obtained by West Virginia personnel during projects in 1982 on which nuclear density gauges were utilized. Here, readings were taken at 4 orientations (0°, 90°, 180°, 270°) at the same location, 3 measurements being taken at each orientation. These data are shown in table 17.

Again, variability is quite low. The average standard deviation for these data sets is 0.4 lb/ft^3 (6 kg/m^3). These multiple reading standard deviations are within the ranges quoted by manufacturers, and indicate that precision of results obtained on concrete should be just as good as precision obtained on materials on which nuclear gauges are in more common use.

In summary, available data indicate that precision of nuclear density gauges, both inherent and under field conditions, is quite good. Average standard deviations for a number of field projects generally fall in the range of 1 to 2 lb/ft^3 (16 to 32 kg/m^3), this figure representing the total test variance, including variability in the concrete itself. Experiments designed to eliminate concrete variability demonstrate the inherent standard deviations to be less than 1 lb/ft^3 (16 kg/m^3), within precision values commonly associated with this technology when applied to other materials.

b. Accuracy of Nuclear Density Gauges

As previously mentioned, development of data relating to accuracy of nuclear density is beset by a number of difficulties. Foremost is the lack of, at least with regards to fresh concrete, a reference method with which the true density of the concrete can be unambiguously determined. While "weighed density" of calibration boxes can be seen as an approximation to true density, the volume of concrete sensed by the gauge does not include the entire volume of the box, and variations in density across the box will contribute more strongly to the gauge reading than to the average value obtained by weighing

Table 16. Results of CMD precision study.[38]

Batch	Mold No.	Average (lb/ft^3)$^{(1)}$	Std. Dev. (lb/ft^3)$^{(1)}$
8	1	148.9	0.9
	2	148.7	0.4
	3	149.0	0.3
9	4	150.6	0.6
	5	149.0	0.5
	6	148.8	0.2
10	7	147.2	0.6
	8	147.8	0.3
	9	149.0	0.3
11	1	148.9	0.7
	2	149.8	0.4
	3	150.4	0.7
12	4	150.9	0.5
	5	150.1	0.3
	6	149.7	0.2
13	7	150.6	0.6
	8	150.0	0.3
	9	150.7	0.2
		Average	0.4

(1) kg/m^3 = lb/ft^{3}x16.018

Table 17. Results of West Virginia precision study - (1982).

Date	Mode	Readings av.$^{(1)}$(std. dev.)$^{(1)}$			
5-08-82	Transmission	139.6(0.5),	138.5(0.4),	135.8(0.1)	-
7-14-82	Backscatter	142.7(0.1),	142.0(0.5),	142.9(0.5),	142.2(0.4)
7-14-82	"	143.1(0.5),	143.5(0.7),	143.4(0.3),	142.5(1.4)
7-14-82	"	142.1(0.2),	142.4(0.3),	142.8(0.2),	142.1(0.2)
7-23-82	Transmission	143.3(0.6),	143.8(0.8),	143.9(0.4),	143.5(0.6)
7-23-82	"	147.1(0.3),	146.4(0.7),	145.3(0.2),	146.1(0.5)
7-23-82	"	143.1(0.1),	143.1(0.3),	143.0(0.3),	143.6(0.4)

(1) kg/m^3 = lb/ft^{3}x16.018

the entire calibration vessel. This effect has been demonstrated by Iddings and Melancon in their studies of twin-probe transmission gauges.[42] Slabs 22- by 12- by 2-inches (6700- by 305- by 50-mm) in dimension were cast from a variety of concretes having a wide range of unit weights. Both weighed densities and nuclear densities (twin-probe) were determined. The distance between the probes was 12 inches (305 mm). Nuclear densities were determined by two groups of operators on the same set of slabs. Difference statistics for the two groups of operators are excellent (Mean difference = 0.1 lb/ft^3 (2 kg/m^3); standard deviation of difference = 1.3 lb/ft^3 (21 kg/m^3)). However, when weighed densities were compared to nuclear densities, the mean difference was 2.4 lb/ft^3 (38 kg/m^3) and the standard deviation of the difference increased to 5.7 lb/ft^3 (91 kg/m^3). Radiographs taken of these slabs after test supported the hypothesis that wide density variations existed throughout the volume of the slab, which would contribute to fluctuation in nuclear gauge readings as compared to the overall, average density of the test slab.

There have been a number of attempts to compare nuclear density readings with densities of core samples extracted from test concrete subsequent to hardening. This approach to estimation of accuracy suffers from at least two drawbacks. First, density of hardened concrete is normally different from density of fresh concrete. Therefore, if nuclear readings are not repeated on the hardened concrete prior to extraction of the core, there will be a systematic error in the comparison. Secondly, the volume sensed by nuclear gauges, either backscatter or transmission, is normally not cylindrical in shape; therefore, the core itself will not include all the same concrete sensed by the gauge, and vice versa. In spite of these limitations, it is instructive to study these comparisons with core densities, as they should represent an upper bound to the deviation to be expected from true density. The early development studies on the CMD included such experiments, which were summarized by the FHWA.[38] Their comparisons are reproduced in this report as table 18.

In spite of the various sources of error noted above, comparisons are quite good. In static conditions, with the CMD held stationary over the test position, the standard deviation of the difference between CMD and cores is less than 1 lb/ft^3 (16 kg/m^3). Under dynamic conditions this variance is increased, but still is very good and approaches the inherent precision of the

Table 18. Laboratory comparison of CMD with conventional test results(38).

CMD as compared with:	Static tests n	Static tests Mean difference[1/] lb/ft^3 [3/]	Static tests Standard deviation[2/] lb/ft^3	Dynamic tests n	Dynamic tests Mean difference lb/ft^3	Dynamic tests Standard deviation lb/ft^3
Weighed density	36	-0.8	0.9	--	--	--
Vibrated unit weight	33	0.5	1.2	72	-1.0	1.5
Commercial nuclear gage density	34	-0.6	0.9	58	-1.7	2.6
Core density	34	0.2	0.8	58	-0.4	1.2
Combination core density (75 percent from top half of core, 25 percent from bottom half)	34	0.5	0.8	58	-0.5	1.2

1/Mean difference $= \dfrac{\sum_{i=1}^{n} (\rho i_A - \rho i_B)}{n}$

where ρi_A and ρi_B = densities by methods A and B, respectively.

2/Standard deviation $= \left[\dfrac{\sum_{i=1}^{n} (\rho i_A - \rho i_B - \text{Mean difference})^2}{n-1} \right]^{1/2}$

3/ $kg/m^3 = lb/ft^3 \times 16.018$

nuclear devices. These data, obtained under well-controlled laboratory conditions by the developers of the equipment, show nuclear gauges to be potentially capable of attaining high accuracy. Table 19 illustrates subsequent lab comparison data obtained using the CMD and a conventional back scatter gauge by Illinois DOT investigators.[39] Here, the accuracy is degraded somewhat as compared to the data presented by the FHWA, standard deviations now being in the range of 2 to 3 lb/ft^3 (32 to 48 kg/m^3).

Results of various field studies where nuclear density readings were compared with core densities generally agree with the second study, as seen in table 20. It should be noted that in the study referred to as ref. 39, air gap compensation had not yet been added to the CMD; this is the reason for the higher standard deviations.

To summarize, while accuracies of nuclear gauges (compared to core density results) under field conditions are generally less than accuracies obtainable under laboratory conditions, absolute differences are within the accepted range of precision for nuclear and conventional density measurements, and variability in comparison statistics (as measured by the standard deviation of differences) is only somewhat higher than standard deviations of nuclear gauge readings obtained under similar field conditions. The conclusion may be drawn that nuclear density gauges (both conventional and CMD) are sufficiently precise and accurate for use in determination of density. At best, precisions can approach those quoted by manufacturers when gauges are used on more homogeneous materials; at worst, precisions are at least as good as could be expected using more conventional approaches such as rodded and weighed densities.

c. Capabilities and Potential Applications of Nuclear Density Gauges

The four basic types of nuclear density gauges, while similar in principle, differ sufficiently in their operations such that their application to various phases of construction carry certain advantages and disadvantages for each gauge type. In order to properly select a gauge for a certain application, a knowledge of these limitations is essential. To offer the reader a ready reference, these advantages and disadvantages have been summarized in table 21.

Table 19. Comparisons of nuclear gauges with conventional test results.(39)

	Tests on Fresh Concrete		Tests on Hardened Concrete	
	CMD vs. Weighed Density	Conventional vs. Weighed Density	CMD vs. Core Density	Conventional vs. Core Density
Number of Readings	9	9	9	9
Mean difference - lb/ft^3 (1)	0.6	-1.6	4.3	0.5
Standard Deviation - lb/ft^3 (1)	2.3	3.1	2.6	2.7

1. $kg/m^3 = lb/ft^3 x16.018$

Table 20. Field comparisons of nuclear and core densities.

Reference	Gauge Type	No. of Readings	Mean Difference (lb/ft^3)(1)	Std. Dev. (lb/ft^3)(1)
(45)	Transmission	10	-0.3	2.2
(39)	CMD	25	-2.0	3.3
(39)	CMD	25	-1.7	4.1
(41)	CMD	21	-0.6	3.1

1. $kg/m^3 = lb/ft^3 x16.018$

Table 21. Relative advantages and disadvantages of nuclear density gauge types.

GAUGE TYPE	ADVANTAGES	DISADVANTAGES
DIRECT TRANSMISSION	Includes full concrete thickness in measurement. Little chemical interference. Widely used in other areas, commercially available. Fairly easy to calibrate Precision is good. Can avoid steel by proper gauge positioning.	Disturbance of concrete. Only measures small volume of concrete placed. Relatively slow measurement. Difficult to clean gauge completely. May be difficult to use where reinforcement is congested. Radiation monitoring required.
BACKSCATTER	Easy to perform. Minimal disturbance of concrete. Widely used in other areas, commercially available. Satisfactory precision. Useful on thin overlays (more sensitive to surface layers). Facilitates cleanup.	Insensitive to deep layers. Volume of influence ill-defined. Sensitive to chemical effects. Reinforcing steel and underlying concrete may interfere. Radiation monitoring required.
TWIN-PROBE	Depth localization of density. Long path length included in measurement. Sensitive to small voids. Little chemical effect. Small radiation source. Good precision.	Increased disturbance of concrete. Slow measurement. Difficult to operate. Equipment not commercially available.
CMD	Large volume of concrete included in measurement. Adaptable to process control. No disturbance of concrete. Satisfactory precision.	Sensitive to surface layers only. Measurement value represents average of volume scanned. Sensitive to chemical effects. Difficult to calibrate. Awkward to handle. Increased radiation hazard. Needs "custom" installation for each paver type. Measurements near pavement edges restricted.

One of the primary considerations in use of nuclear density gauges is the volume of material which the particular gauge includes in the measurement. In some cases, such as full depth paving, it may be desirable to obtain an average reading for the entire thickness of pavement. In other applications, such as thin overlays, only the density of the overlay is of interest and, therefore, a gauge sensitive to surface layers only is desirable. Direct transmission gauges offer the capability of including the full thickness, up to 10 inches (250 mm), in the measurement. Backscatter gauges may be more appropriate to thin overlays, as they are most sensitive to the density included in the surface layers. If, in addition to a density averaged across the depth, localization of density with depth is desired, then the twin-probe gauge would be a more appropriate choice. This would afford the user the opportunity to evaluate density above and below reinforcing mesh or continuous reinforcement, and thereby obtain information on the ability of the consolidation equipment being used to consolidate concrete below the reinforcement.

The volume of concrete measured in relation to the total volume placed is also an important consideration, especially where nuclear density readings are to be used in decisions relating to acceptance of concrete. The major disadvantage of the "static" types of gauges, i.e., direct transmission, backscatter, and twin-probe, is that they basically constitute a point measurement, as compared to the total volume of concrete placed. In order to obtain meaningful statistics in full-scale paving operations, a very large number of measurements would be needed. This may prove to be impractical where gauges requiring insertion into the concrete are used (transmission and twin-probe), and may also be difficult with backscatter gauges, unless a large commitment in operators and equipment is made. The CMD has an obvious advantage in this regard, as large areas of concrete can be measured in real time during the paving process. The disadvantage lies in the fact that the CMD is basically a backscatter instrument; therefore little or no information is supplied with regards to densities below 4 inches (100 mm) from the surface. One means of overcoming this limitation would be to supplement CMD readings with direct transmission measurements at selected points. These points could be chosen in areas where CMD measurements indicated that surface densities were low, or at randomly selected points in the newly-placed pavement.

Another area where comparisons are useful relates to the general handling and operating characteristics of the various gauge types. Perhaps the easiest to operate is the backscatter type. Here no penetration of the concrete is required, and there is no source rod to clean. This is especially important with regards to radiation safety, as wiping of the source rod can, in theory, lead to excessive radiation exposure if done over long periods of time. The most difficult to operate are the twin-probe and CMD. The twin-probe requires more exact positioning of the gauge, and one must ensure that the concrete surface is fairly smooth so that the probes penetrate the surface in a perpendicular geometry. The CMD is rather heavy, of the order of 180 lb (85 kg); this makes calibration somewhat difficult and awkward, and increases the time needed both for setup of the equipment on the paver and subsequent cleanup after the operations have been completed.

One other important area where gauge performance comparisons are instructive relates to the effect of mesh, dowel bars, and reinforcing steel on gauge response. Provided that steel is positioned below the surface layers of the pavement, backscatter and CMD gauges will be little affected by its presence. Mitchell reported that No. 6 (0.19 inch, 4.9 mm) mesh positioned at 2 inches (50 mm) or No. 6 (3/4 inch, 19 mm) bar positioned at 3 inches (80 mm) had little effect on density readings of the CMD.(38) Spalti and Lemar, using a commercial backscatter gauge, found that No. 5 (5/8 inch, 16 mm) bar had little effect on readings when located greater than 2.5 inches (64 mm) below the slab surface, but added approximately 4 to 5 lb/ft^3 (64 to 80 kg/m^3) to readings when located 1.5 inches (38 mm) from the surface.(47) In the 1977 survey, California noted that there was little effect of reinforcing steel when located 4 inches (100 mm) below the surface, but densities increased by 1.5 and 5.3 lb/ft^3 (24 and 85 kg/m^3) when steel depth was decreased to 2.5 inches (64 mm) and 1.5 inches (38 mm), respectively.(43)

When steel is located relatively close to the surface, yet deep enough (usually 1 to 2 inches (25 to 50 mm)) to allow operation in the direct transmission mode, more accurate results may be obtained using direct transmission readings. However, as previously noted, direct transmission is somewhat slower, and cleanup is more tedious due to the need for cleaning of the source rod.

Direct transmission readings over thicknesses which encompass embedded steel will be influenced by the presence of the metal. The magnitude of the effect is determined by the size and spacing of the steel. For instance, a mat of No. 5 (5/8 inch, 16 mm) bars on 4-inch (100 mm) centers with 1.5 inches (38 mm) of cover had a large effect on transmission readings taken at the 2-inch (50 mm) depth, increasing average density by 6 lb/ft^3 (96 kg/m^3) over specimens containing no steel.[47] However, No. 5 (0.20 inch, 5.2 mm) wire mesh placed at a depth of 3 inches (75 mm) produced very little influence on direct transmission readings at 4-inch (100 mm) depth.[45] These data indicate that, while the relatively thin gauge welded-wire mesh will not significantly interfere with results, reinforcing steel bars can have a large influence, and the positioning of the source rod should be determined by testing prior to use of direct transmission measurements on reinforced concrete.

The application of nuclear density gauges to highway construction should afford both contractors and inspectors with improved capabilities for process control and acceptance testing. The capabilities of the four basic gauge types are summarized in table 22.

Nuclear density gauges allow a direct measurement of vibrator performance and efficiency of consolidation to be made. It is unlikely that sufficient readings could be obtained with static gauges so that each vibrator on a paver could be monitored; in this case the CMD would be more appropriate. However, studies by Texas Transportation Research Institute[21] have shown that problems do exist with consolidation of the lower portions of a concrete slab. In this case, while the CMD would allow operators to assess whether vibrators were functioning sufficiently to consolidate the top layers of the slab, periodic checks should be made either with a direct transmission or twin-probe gauge to determine if consolidation is adequate in the bottom layer. If two gauges were available, it is estimated that the readings could be obtained approximately every 30 feet (9 m) assuming the paver were traveling at 15 ft/min (4.5 m/min) and 2 minutes were required for the readings. Static readings would be more useful in areas where hand-held vibrators are often used to ensure that concrete is consolidated, such as at construction joints, around dowels, and dowel basket assemblies.

Nuclear density gauges should also prove a useful tool in acceptance testing. Currently, provided that thickness specifications are met and the pavement is not obviously honeycombed or defective, no tests are routinely

Table 22. Capabilities of nuclear density gauges.

GAUGE TYPE	PROCESS CONTROL		ACCEPTANCE TESTING	DETECTION OF MIX DEFECTS
	Vibrator Operations	Adequacy of Consolidation		
Direct Transmission	Multiple readings required for paver units. More applicable to areas where hand vibrators used.	Can include entire thickness in measurement. Accurate within 2 lb/ft^3 (32 kg/m^3). Readings can be obtained at various depths (but nonlocalized). Spot readings near joints and mesh can be obtained.	Average for thickness in measurement. To obtain representative measurement across pavement multiple readings would be required.	Average air content variations ($\pm$2 percent) over thickness. Detection of water content variations of $\pm$15 percent possible.
Backscatter	Multiple readings required for paver units. Could apply to hand held units, but depth limited.	Most useful for thin overlays. Surface layers up to 2-in (50 mm) contribute most heavily to readings. Little disturbance of concrete during test.	Somewhat more rapid measurement if multiple readings required. Precision somewhat less than with direct transmission. Surface could be treated separate from bulk pavement.	Could detect air content in surface layer to $\pm$2 percent. Could detect water content variations in surface layer to $\pm$15 percent.
Continuous (CMD)	Verify vibrator operation by continuous scan. Vibrator efficiency evaluated in top layer only. Cannot penetrate beyond 4 in (100 mm).	Offers rapid scan of pavement surface. Speeds up to 17 ft/min (.085 m/s) obtainable. Can penetrate up to 4-in(100 mm) into pavement. Continuous readout obtained.	Large, representative volume is measured. Real-time readout available. Volume included adjustable by change in time constant.	Mix defects averaged over volume measured.
Twin Probe	Multiple readings required for paver units. Possibility of indicating depth of vibrator influence.	Offer localized density readings with depth. Can locate void areas.	Localized acceptance possible. Location of void areas at depth possible.	Potential for localization of areas of high water or air content.

performed which assure the owner that proper consolidation has been achieved. While many other variables may contribute to the ultimate quality and performance of concrete construction, if the consolidation is poor, strength will suffer, permeability will increase, and the service life will most likely be shortened. While the use of nuclear density gauges is certainly desirable from an acceptance viewpoint, limitations on sampling restrict use of many of the gauge types. Even with a number of gauge operators active at one time, the total volume of pavement which can be inspected is only a fraction of the total amount of concrete actually placed. While this might not be a serious limitation if defects were to occur in a uniform manner, most often those areas where consolidation is poor occur only infrequently; therefore the likelihood that point readings would enable inspectors to detect these areas is small. This limitation is not as great in handworked areas, where the total volume of concrete placed is much less than in mainline paving. These limitations on static gauges would appear to make the CMD the choice for monitoring of straightaway pavement consolidation, with static gauges used for checks in questionable areas and at joints, and for information on variation of density with depth.

There are some possibilities for use of nuclear gauges for detection of mix defects other than incomplete consolidation. A rough calculation shows that the inherent precision of nuclear density gauges will allow for detection of variation in air content of ± 2 percent, and variations in water content (of a typical mixture) within ± 15 percent, at best. However, this assumes that the density is not changing for other reasons at the same time. If this can be verified, then nuclear density gauges may offer a means of controlling, at least in a qualitative manner, large variations in mix characteristics such as air and water contents.

QUALITY CONTROL PROCEDURE DEVELOPMENT

This section addresses the development of quality control procedures based on degree of concrete consolidation. Because a concrete pavement is a manufactured item, it is necessary to ensure that quality control is exercised at all critical phases of production. Presently, quality control specifications exist to assure a quality product through all phases, up to delivery of concrete at a site. <u>However, there are no direct specifications available to monitor concrete quality after the concrete is placed in front of the paver</u>. As shown by the results of the present study, concrete properties and therefore pavement performance can be greatly affected by the degree of consolidation. Inadequate consolidation can result in concrete that is neither strong nor durable and consequently can result in premature failure or loss of serviceability of the pavement.

Acceptance sampling is a systematic procedure for deciding for a given level of risks whether to accept or reject the product inspected. A good sampling plan forces the supplier to control the quality of his product. A very strict acceptance sampling plan can be unfair to the supplier and may result in an unnecessarily higher price for the product. Thus, a balance has to be maintained between an acceptable level of quality of the product and the price of the product.

1. <u>Existing Specifications for Paving Concrete</u>

Existing specifications for paving concrete can be classified into two broad categories. The first category relates to the strength of hardened concrete, and the second category relates to the properties of the plastic concrete delivered to the site. In addition to specifying the quality of concrete, specifications are also used to ensure the quality of the finished pavement by specifying tolerances for pavement thickness and the finished surface characteristics.

a. Specifications for Plastic Concrete

Specifications for plastic paving concrete generally include the following three items:

- Unit weight.
- Slump.
- Air content.

Unit weight of paving concrete is measured primarily to control uniformity of the concrete within a batch and from batch to batch. The weight per cubic foot of concrete is determined according to the procedures of ASTM Designation: C138 (Standard Test Method for Unit Weight, Yield, and Air Content (Gravimetric) of Concrete). The use of this test is not very common for paving concrete. The test is specified when low slump concrete is used for bridge deck construction.

Slump is a test to measure the consistency of concrete and the test is made in accordance with ASTM Designation: C143 (Slump of Portland Cement Concrete.) For paving concrete, slump is specified to control the potential strength of concrete and to reduce the risk of edge slumping when slipform paving is used. For vibrated slipform paving, slump is generally specified to be between 1/2 inch and 1-1/2 inches (13 and 38 mm). For machine finished, fixed-form pavement, slump is generally specified to be between 1-1/2 and 2-1/2 inches (38 and 64 mm). Slump tests should generally be made with a frequency of not less than one test for each 150 yd^3 (115 m^3) of concrete or at a rate of one test for each 200 ft (60 m) of two-lane paving. The truckload of concrete not meeting the specified limits for slump is rejected.

Air content of concrete is specified to ensure concrete durability. Air content is measured using the procedures of ASTM Designation: C173 - Test Method for Air Content of Freshly Mixed Concrete by the Volumetric Method or ASTM Designation: C231 - Test Method for Air Content of Freshly Mixed Concrete by the Pressure Method. The air content specified is generally based on the nominal maximum size of aggregate in the mixture and the exposure condition. For example, the air content specified for concrete with 1-1/2 inch (38 mm) maximum aggregate size in severe exposure condition is 5.5 to 6.0 percent. A tolerance of ± 1.5 percent of the specified value is generally allowed. Air content and slump tests are generally made on the same batch of concrete at a frequency of not less than one test for each 150 yd^3 (115 m^3) of concrete delivered to the site.

Although, statistically based acceptance sampling plans can be developed for slump and air content tests, most highway agencies accept or reject concrete on the basis of a truckload only, i.e., if a test fails, only that single truckload is rejected.

b. Specifications Based on Strength

Acceptance of concrete based on strength is a very common and accepted procedure. Strength may be defined in terms of compressive or flexural strength. Many highway agencies prefer to use compressive strength even though concrete pavement design is usually based on flexural strength. Many agencies, however, do not specify a minimum strength. These agencies attempt to control the quality of concrete by specifying the mix design and the quality of the concrete-making materials.

A major problem with controlling the quality of concrete by specifying a minimum strength is that test results are not available until 28 days after concrete placement. In addition, because the specimens used for strength tests are molded, the quality of these specimens is not necessarily representative of the quality of concrete which is greatly influenced by the degree of consolidation.

When concrete strength is specified, it is usually specified as follows:

> Concrete represented by a strength test of at least 95 percent of the required 28-day compressive strength will be acceptable for cast-in-place and precast concrete.(48)

or

> The average of any n consecutive strength tests, tested at the end of 28 days, shall have an average strength equal to or greater than the specified strength.(49)

The American Concrete Institute's (ACI) Recommended Practice for Evaluation of Strength Test Results of Concrete (ACI 214-77) describes four different methods of specifying concrete strength, including the two examples given above. However, ACI recommends use of an associated probability for each method.

Highway agencies have usually not used pay schedules for concrete strength, although pay schedules are widely used for pavement thickness. The Federal Aviation Administration provides for acceptance of concrete based on a pay schedule for each lot of concrete. This pay factor schedule is given below.(50)

Pay Factor	Acceptance Limits Average Flexural Strength (4 tests)
1.00	greater than M+0.120R
0.95	M to M+0.115R
0.70	M-0.090R to M-0.005R
0.50	less than M-0.090

where: M = specified 28-day flexural strength
R = range of sample of size n = 4

Weed has suggested use of pay schedules for concrete used for highway paving.(51,52,53,54) Weed suggests using pay schedules for withholding sufficient payment at the time of construction to cover the extra cost anticipated in the future as the result of deficient-quality work.(52)

2. Effect of Consolidation on Pavement Service Life

As part of the present study, laboratory data was developed on the effect of consolidation on some of the important properties of concrete. These data were presented earlier in the report. Data were developed for the following properties of concrete:

- Compressive strength.
- Bond of reinforcing steel to concrete.
- Permeability of concrete to chloride ions.
- Resistance of concrete to freezing and thawing in water.

a. Effect of Consolidation on Strength

Laboratory test data indicate that the degree of consolidation has a significant effect on compressive strength. Test results show that compressive strength is reduced by about 30 percent for each 5 percent decrease in degree of consolidation. The general relationship between percent consolidation and compressive was best described by the form of equation 3. The relationship for compressive strength is given below:

$$f_{cx} = 10^{2-[A_1(100-x)]} \quad (10)$$

where: $f_{cx} = f_{c,x}/f_{c,100}$

$f_{c,x}$ = compressive strength at x percent consolidation

$f_{c,100}$ = compressive strength at 100 percent consolidation

A_1 = regression constant (from table 8)

Since pavement analysis and design procedures generally use flexural strength (modulus of rupture), the consolidation/compressive strength relationship was used to establish the relationship between flexural strength and degree of consolidation.

The flexural strength, f_r, is generally estimated from the compressive strength, f_c, using the following relationship:

$$f_r = K(f_c)^n \tag{11}$$

where: K = a constant

n = 0.5

Substituting equation 11 into equation 10, the following is obtained:

$$f_{rx} = 10^{2-[0.5A_1(100-X)]} \tag{12}$$

where:

f_{rx} = $f_{r,x}/f_{r,100}$

$f_{r,x}$ = flexural strength at x percent consolidation

$f_{r,100}$ = flexural strength at 100 percent consolidation

The values of the coefficient A_1 are given in table 8 for the six different mixes used for the study. The value of A_1 when all the data for the six mixes were combined equals 0.0364, with a standard error of estimate of 0.0602 applicable to the logarithmic form of equation 10. Equation 10 has a coefficient of correlation of 0.965. The equation for the combined data can be written as follows:

$$f_{rx} = 10^{2+0.5\,[-A_1(100-X)\pm Z(0.0602)]} \tag{13}$$

where: Z = standard deviate associated with a given level of confidence limits

= 1.96 for 95 percent confidence limits

Equation 13 is tabulated in table 23. The effect of degree of consolidation on projected service life of a concrete pavement can be estimated using equation 13 and the procedures given in the AASHTO's "Guide for Design of Pavement Structures." The AASHTO design guide contains an equation that predicts the number of equivalent 18-kip single-axle loads (SAL) that a concrete pavement can sustain as a function of several design parameters such as concrete flexural strength and pavement thickness.[15]

The projected service life of a concrete pavement was determined as a function of degree of consolidation using equation 13 and the AASHTO design equation. The projected service lives are given in table 24. It is seen that concrete consolidation can have a significant effect on pavement service life. At 90 percent consolidation, projected pavement service life is only 5.4 years.

The effect of normal material variability associated with the construction process should also be considered in the analysis of the effect of consolidation on pavement service life. For simplicity, it is assumed that the measurements related to the degree of consolidation can be considered to be normally distributed. Then, the effect of normal material variability on pavement performance can be considered by assuming the following conditions:

Measured average degree of consolidation, $u = 98$ percent
Measured standard deviation due to material variability,
$\sigma_m = 1.0$ percent
Design flexural strength = 600 psi (4.1 MPa)

Then the expected service life for a pavement whose average degree of consolidation is 98 percent can be computed as shown in table 25 and is computed to be 16.5 years. The expected service lives for measured average degrees of consolidation of 99 and 100 percent are 18.4 and 19.6 years, respectively. In the above computations, it was assumed that the measured flexural strength at 100 percent consolidation equaled the design flexural strength. In practice, the measured average flexural strength on properly controlled projects is considerably higher than the design strength. This is because, on many projects, concrete acceptance is also based on acceptance criteria based on strength. These strength-based plans allow for only a small fraction of tests to fall below the specified design strength. The effect

Table 23. Effect of consolidation on concrete flexural strength.

Percent Consolidation, x	$f_{r,x}/f_{r,100}$		
	Lower Limit	Average	Upper Limit
80	0.38	0.44	0.49
82	0.41	0.47	0.54
84	0.45	0.51	0.59
86	0.49	0.56	0.64
88	0.53	0.61	0.69
90	0.57	0.66	0.75
92	0.62	0.71	0.82
94	0.68	0.78	0.89
96	0.74	0.85	0.97
98	0.81	0.92	1.05
100	0.87	1.00	1.15

<u>Notes</u>: 1. Based on Equation 13.
2. Upper and lower limits refer to the 95 percent confidence limits.
3. $f_{r,x}$ = flexural strength at x percent consolidation.
$f_{r,100}$ = flexural strength at 100 percent consolidation.

Table 24 Effect on consolidation on projected service life.

Percent Consolidation	Projected Service Life, years
100	20.0
99	17.6
98	15.4
97	13.5
96	11.9
95	10.4
94	9.1
93	8.0
92	7.0
91	6.2
90	5.4

<u>Notes</u>:

1. Straight-line projection for traffic growth assumed.
2. Design concrete flexural strength (at 100 percent consolidation) = 600 psi (4.1 MPa).
3. Projected service life is independent of design pavement thickness.

Table 25. Expected service life.

Percent Consolidation	Probability of Occurrence, % (1)	Service Life, years Projected (2)	Expected (3)
>100	2.3	20.0	0.46
99-100	13.6	20.0	2.72
98- 99	34.1	17.5	5.97
97- 98	34.1	15.3	5.21
96- 97	13.6	13.8	1.88
95- 96	2.2	12.0	0.26
94- 95	0.1	10.4	0.00
		Expected Service Life	16.50

<u>Notes</u>:

1. Probability of occurrence estimates the fraction of the pavement with the given degree of consolidation.
2. Projected service life is the service life predicted if all of the pavement was consolidated at the given level of consolidation.
3. Expected service life is the contribution of each fraction of the pavement to the overall pavement service life and is obtained by multiplying the values in the second and third columns.

of the degree of consolidation on inplace flexural strength relative to the design strength for different values of flexural strength at 100 percent consolidation is shown in table 26. Thus, if the concrete supplied at the site has an average strength at 100 percent consolidation greater than the design strength, the expected service life as given in table 25 would be higher. The expected service life for such a case is given in table 27.

b. Effect of Consolidation on Other Properties of Concrete

The effect of consolidation on bond of reinforcing steel to concrete and on permeability of concrete to chloride ions was also found to be significant. As discussed earlier, there is a loss of approximately 50 percent in bond strength for a 5 percent reduction in the degree of consolidation. Average bond strength measured at 100 percent consolidation was about 1000 psi (6.8 MPa), and therefore the bond strength developed at 95 percent consolidation has a value of about 500 psi (3.4 MPa).

Bond development is primarily of importance to continuously reinforced concrete pavements (CRCP). As discussed in section 2, bond strength influences the minimum crack spacing that need to be developed in CRCP; also a minimum bond strength of about 300 psi (2.1 MPa) must be available. Therefore, it would appear that an absolute lower bound for degree of consolidation for CRCP would need to be at least 95 percent to ensure effective bond between the reinforcing and the concrete and that a desirable lower bound of about 97 percent would be ideal.

There is no direct method to evaluate the effect of permeability of concrete to chloride ions on pavement performance. There have been few reported instances of deterioration of CRCP due to corrosion of reinforcing steel caused by deicing salts. However, in these instances, salt solutions penetrated through cracks which normally form in CRCP down to the level of the steel, and not through the concrete itself. While results of the present study show that poor consolidation can greatly increase permeability of the concrete, it is likely that penetration through open cracks would still be greater than penetration through even a poorly consolidated concrete, and corrosion would still be initiated at the cracks. It is possible that poor consolidation around reinforcing bars would accelerate the lateral penetration of salts once the salt had reached the steel and thereby increase the rate of deterioration of the steel and, ultimately, the overlying concrete. This

Table 26. Inplace flexural strength relative to the design strength.

Percent Consolidation	$f_{r,x}/f_{rd}$	
	$f_{r,\ 100} = f_{rd}$	$f_{r,\ 100} = 1.1\ f_{rd}$
80	0.44	0.48
82	0.47	0.52
84	0.51	0.56
86	0.56	0.62
88	0.61	0.67
90	0.66	0.73
92	0.71	0.78
94	0.78	0.86
96	0.85	0.94
98	0.92	1.01
100	1.00	1.10

f_{rd} = design flexural strength
$f_{r,x}$ = inplace flexural strength at x percent consolidation
$f_{r,100}$ = flexural strength at 100 percent consolidation

Table 27. Projected service life for different values of flexural strength at 100 percent consolidation.

Percent Consolidation	Projected Service Life, Years	
	$f_{r,100} = f_{rd}$	$f_{r,100} = 1.1f_{rd}$
100	20.0	>20.0
99	17.6	>20.0
98	15.4	>20.0
97	13.5	18.2
96	11.9	16.3
95	10.4	14.0
94	9.1	11.9
93	8.0	11.0
92	7.0	8.2
91	6.2	7.0
90	5.4	6.5

f_{rd} = design flexural strength
$f_{r,100}$ = flexural strength at 100 percent consolidation

problem can only be approached qualitatively; however, to the investigators' knowledge, there have been no controlled studies of such phenomena which would offer guidance as to rate of progress of such deterioration and its effect on service life of CRCP.

Results of the present study show that consolidation has only minor effects on the resistance of concrete to freezing and thawing, provided the concrete is of good quality and properly air-entrained. It is possible that poor consolidation may have a greater impact on pavement durability in actual pavements
in the presence of traffic loading.

c. Summary

As discussed in the preceding sections, methods exist to relate the effect of degree of consolidation to pavement service life based on only the strength characteristics. Thus, in the development of acceptance plans for concrete consolidation, only the effect of consolidation on strength and thereby on expected pavement service life is considered. It is assumed that if the strength-based acceptance criteria are satisfied, then there will be sufficient assurance that the other characteristics such as steel/concrete bond strength and permeability to chloride ions would be acceptable.

3. <u>Development of Acceptance Plan Based on Consolidation</u>

The development of an acceptance plan requires consideration of at least the following items:

- Risks involved for the buyer and the seller.
- Quality levels - the acceptable quality level (AQL) and the rejectable quality level (RQL).
- Operating characteristic curves.
- Lot size and sample size.

a. Consideration of Buyer's and Seller's Risks

When a sampling plan is used, a decision has to be made to accept or reject the material in question. Two types of risks are involved with each decision. These risks are usually termed the buyer's risk and the seller's risk. The buyer's risk, denoted as β, is the risk of erroneously accepting unsatisfactory material. The seller's risk, denoted as α, is the risk of erroneously rejecting satisfactory material. The values generally selected for α and β are based on the criticality of the material or process to the final performance of the product, the product being the concrete pavement in our case. The following are historically recommended risk levels for different levels of criticality.

Classification of Criticality	Seller's Risk (α)	Buyer's Risk (β)
Critical	5.0%	0.5%
Major	1.0%	5.0%
Minor	0.5%	10.0%
Contractual	0.1%	20.0%

Uninterrupted use of highway facilities is considered to be of prime concern to highway agencies. Therefore, any element of the construction process that directly affects pavement performance should be considered as a major item. As already discussed, concrete consolidation can have a significant effect on pavement performance.

Therefore, concrete consolidation is considered a major item. For the purpose of development of the model acceptance sampling plans, the following risks were selected:

Buyer's risk (β) = 5 percent

Seller's risk (α) = 5 percent

The above risk levels are commonly used in highway construction.

b. Acceptance Plan for Inspection by Variables

Acceptance plans for inspection by variables are used when the characteristics of the product being inspected can be measured. Variable plans are more statistically efficient than attribute plans and provide greater discriminating power for a given sample size. A variable plan requires computation of the mean and the standard deviation for a given lot and requires selection of a sample size per lot.

In developing a variable acceptance plan for concrete consolidation, procedures outlined in the "Interim Recommended Practice for Acceptance Sampling Plans for Highway Construction (AASHTO Designation: R9-85)", were followed.[55] The use of a variable acceptance plan requires that the population being sampled is normally distributed. Data on densities of concrete in highway paving indicate that the assumption of normality of density measurements is reasonably valid.

(1) <u>Desirable Acceptance Values for Degree of Consolidation</u>. The desirable acceptance values (DAV) were calculated for the following conditions:

Seller's risk, α = 5 percent

Mean values of degree of consolidation attainable on well constructed projects, X = 100, 99, 98 percent

Standard deviation of the population of measurements of the degree of consolidation, σ = 1.0, 1.5, 2.0 percent

Sample size (number of tests) per lot = 5, 10

The desirable acceptance values (DAV) are given in table 28. The desirable acceptance value is the value below which only α percent of data points are expected to fall. If it is assumed that, on well controlled projects it is reasonable to expect 98 percent consolidation as the population mean and a population standard deviation of 1.5 percent, then the DAV would equal 96.9 and 97.2 percent for sample sizes of 5 and 10, respectively. If the DAV of 96.9 were to be considered the lower limit value for acceptance based on sample size of 5, then based on table 27 it can be stated that 95 percent of the pavement would probably have a service life at least 13.0 years for $f_{r,100} = f_{rd}$ or at least 18.0 years for $f_{r,100} = 1.1\ f_{rd}$.

Sufficient data are not available from field projects to enable reliable estimation for the population mean, $\bar{X}$, and the population standard deviation, σ. Also, in many instances when density data are available, reference unit weights are not available. Thus, in such cases it is not possible to calculate the degree of consolidation. However, the limited data that are available, such as presented in table 12 of this report, indicate that, on well constructed projects it is possible to achieve a mean degree of consolidation of at least 98 percent of the rodded unit weight. Table 12 also indicates that the average overall standard deviation obtained when consolidation is measured using a nuclear gauge is about 2.0 pcf (32 kg/m^3) or about 1.4 percent of the rodded (reference) unit weight.

In practice, an estimate of the population mean can be obtained by averaging the mean of the degree of consolidation measured from several projects. Similarly, a reliable estimate of the population standard deviation can be obtained by calculating the pooled standard deviation from data obtained from several projects. The pooled standard deviation is computed as follows:

Table 28. Desirable acceptance values for degree of consolidation.

Sample Size (n)	σ,%	$\bar{\bar{X}}$,%	DAV
5	1.0	100	99.3
		99	98.3
		98	97.3
	1.5	100	98.9
		99	97.9
		98	96.9
	2.0	100	98.5
		99	97.5
		98	96.5
10	1.0	100	99.5
		99	98.5
		98	97.5
	1.5	100	99.2
		99	98.2
		98	97.2
	2.0	100	98.9
		99	97.9
		98	96.9

Notes: $\bar{\bar{X}}$ = population mean
σ = population standard deviation.
DAV = desirable acceptance value

$$s_p^2 = \frac{(n_1-1)s_1^2 + (n_2-1)s_2^2 + \ldots .(n_k-1)s_k^2}{n_1 + n_2 + \ldots . + n_k - k}$$

where: s_p = pooled standard deviation (unbiased estimate of σ)
s_i = standard deviation at project i having a sample size n_i
k = total number of projects

For the purpose of development of a model sampling plan based on consolidation, the following criteria are used:

Achievable standard deviation for the population of degree of consolidation measurements = 1.5 percent

Lower limit for acceptance of degree of consolidation measurements = 97 percent.

(2) Operating Characteristic Curves. Operating characteristic (OC) curves were developed for acceptable quality level (AQL) of 10 percent defective. The AQL is that level of lot percent defective at or below which the work is considered to be completely acceptable. These OC curves were selected to ensure that the seller's risk was about 5 percent. Thus, the probability of accepting a lot with an AQL of 10 percent is 95 percent.

Operating characteristic curves for sample sizes of 5, 10, and 15 are shown in figure 26 for the information given in table 29 obtained from reference 55. Use of OC curves requires knowledge of the quality index, Q_L, and the acceptance constant, k. The relationship between Q_L and k is given below:

$$Q_L = \frac{\bar{X}-L}{S} \geq k \tag{15}$$

where: $\bar{X}$ = sample mean
S = sample standard deviation
L = lower specification limit outside of which the material is defined to be defective.

The three OC curves shown in figure 26 satisfy the requirement that the probability of accepting a lot that is 10 percent defective (AQL) is 95 percent. The primary difference in the three curves is that as the sample size is reduced, the probability of accepting a defective lot increases. Thus, if the rejectable quality level (RQL) of percent lot defective was set

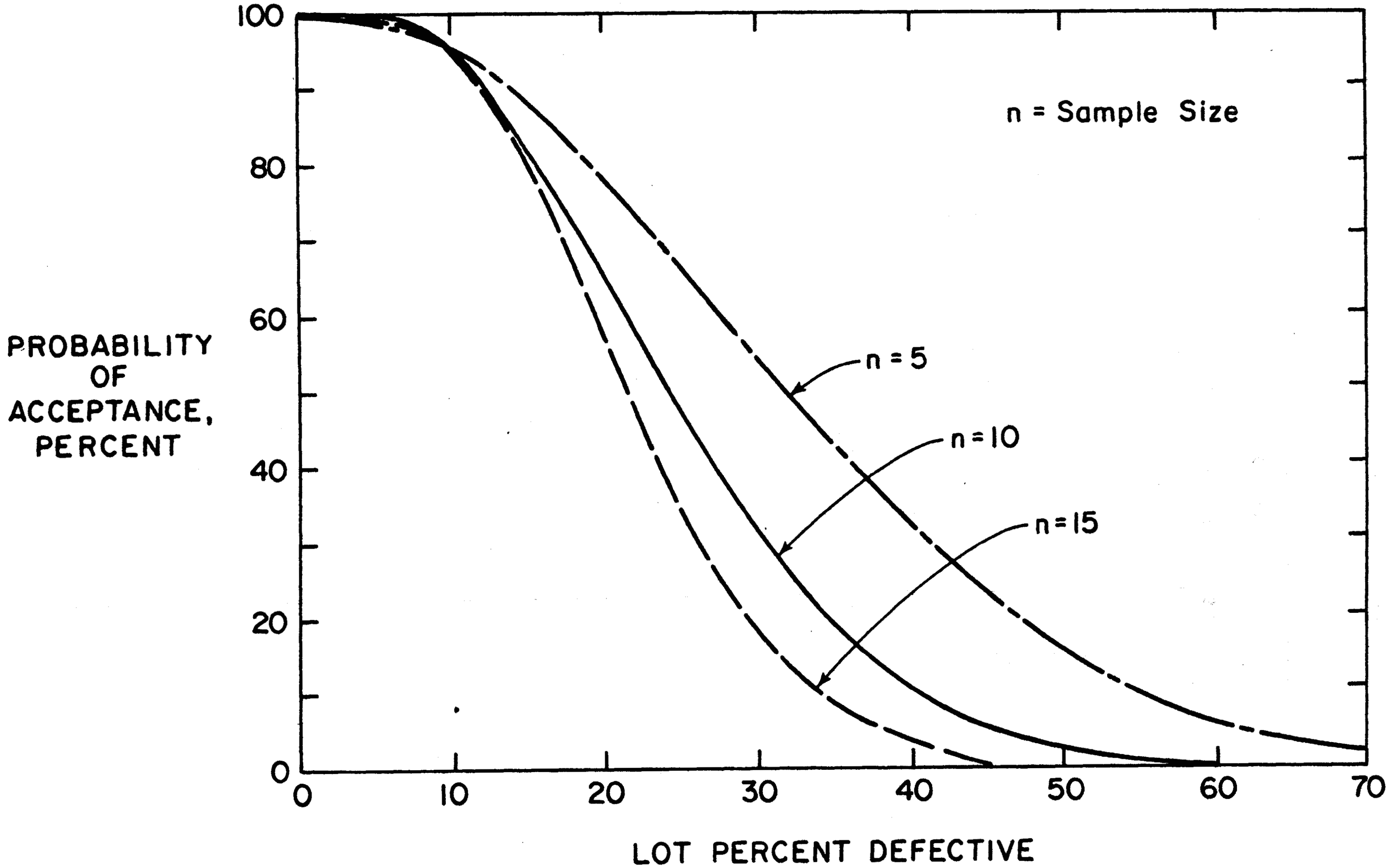


Figure 26. Operating characteristic curves.

Table 29. Operating characteristic curve data (from ref. 55)

Lot Percent Defective	Probability of Acceptance		
	n = 5 k = 0.51	n = 10 k = 0.72	n = 15 k = 0.81
10	95	95	95
20	77	65	56
30	54	31	18
40	32	10	-
50	16	2	-
60	6	0	-
70	2	0	-

k = minimum value of quality index, Q_L, for the lower specification limit

Q_L = quality index for the lower specification limit

$$= \frac{\bar{X}-L}{S}$$

$\bar{X}$ = sample mean

S = sample standard deviation

L = lower specification limit outside of which the material is defined to be defective at 50 percent, then the probability of accepting at RQL would be 16, 2, and 0 percent, respectively, for sample sizes of 5, 10, and 15. If the probability of accepting at the level RQL is to be as low as possible, a sampling plan using a sample size of 10 would be regarded as satisfactory. If the probability of accepting at the RQL level is set at 5 percent, then a sampling plan using a sample size of 8 would be regarded as satisfactory. The OC curve for a sample size of 8 is similar to that for sample size of 10.

When sample size of 8 is used, the following condition would need to be satisfied.

$$Q_L = \frac{\bar{X}-L}{S} \geq k = 0.66 \qquad (16)$$

The application of a sampling plan to inspection of consolidation can now be considered. For the model acceptance plan, the lower limit for acceptance of degree of consolidation is established at 97 percent. Then, if a sample size of 8 is used with a corresponding k value of 0.66, the following sample mean would be required as a minimum for the lot to be considered acceptable:

Sample Standard Deviation, S, percent	Sample Mean, $\bar{X}$, percent
1.0	97.7
1.5	98.0
2.0	98.3

The maximum allowable estimated percent lot defective, M, associated with sample size of 8 and k of 0.66, is 26.

4. Model Acceptance Plan

The model acceptance plan presented in this section is applicable when use is made of portable nuclear density gauges. Discussion of the application of the CMD to evaluation of the degree of consolidation is given in section 6. The model acceptance plan for degree of consolidation is based on $\alpha = \beta = 0.05$ and can be stated as follows:

Take n = 8 randomly located measurements to determine the degree of concrete consolidation. Compute the sample mean, $\bar{X}$, and the sample standard deviation, S. Then, use $\bar{X}$ and S to compute the quality index, Q_L, as follows:

$$Q_L = \frac{\bar{X}-97}{S} \qquad (17)$$

For the lot to be judged acceptable, Q_L should be equal to or greater than k = 0.66.

A lower limit for the degree of consolidation of 97 percent was used for the model plan. This lower limit for the degree of consolidation should be as high as is achievable in the field. It is expected that as more experience is gained with acceptance sampling based on degree of consolidation, a better estimate for the lower limit would be established.

It should be noted that although AQL is considered to be 10 percent defective, the acceptance procedure must allow the sample estimate of percent defective to be as large as M = 26. This allowance takes into account sampling variability. Thus, when the lot is truly 10 percent defective, there is still a probability of 5 percent that the estimated percent defective will be equal to or greater than 26.

a. Lot Size

An essential requirement in selecting the lot size is that it represents uniformity in terms of source of concrete and the placement procedure used. With respect to selecting a lot size for performing density tests, the lot size should be such that the required number of tests can be performed without disrupting the concrete placement or finishing operations.

Assuming that a single reading with a 1-min time period is used with the nuclear density gauge and that another five minutes are required for setup and cleanup, then the minimum lot size for a sample size of 8 is obtained as follows:

Paver Speed, ft/min (m/min)	Lot Size (length), ft (m)
5 (1.5)	240 (75)
10 (3)	480 (150)
15 (5)	720 (225)

A width of paving of 24 ft (7.3 m) is assumed. The lot size (length) can of course be decreased if more than one nuclear gauge is used. However, this would also result in increased testing costs. As more experience is obtained in the field for routine use of nuclear gauges, the lot size can be appropriately selected such that the testing poses no delays to the paving operation.

b. Test Locations

The eight test locations per lot should be selected randomly. A simple method to do so is to divide the lot into 8 sublots. The test would then be performed at a randomly located point at midlength of the sublot. The test point is randomly selected by dividing the paving width in 1-ft (0.3 m) increments not counting the 1-ft (0.3 m) width at each edge. These increments are numbered sequentially starting with 1 from the first increment near the shoulder (outside) edge. Thus, for a 24-ft (7.3 m) paving width, there will be 22 increments. Then, a random number generator (calculator or tabular) is used to select the increment number for the test location. The outlined procedure allows forward progress at a uniform rate. However, any other statistically reliable procedure can be used for selecting test locations.

c. Retesting

For the model acceptance plan, it is suggested that retesting be permitted when the lot is considered as not being acceptable. Retesting constitutes a sequential acceptance scheme with a larger (double) sample size. Thus, a different acceptance constant, k, would need to be established to ensure that the probability of acceptance at AQL and RQL are acceptable.

For the model plan presented based on sample size of 8 and AQL of 10 percent defective, approximately 5 percent of the lots would be found unacceptable when degree of concrete consolidation is at AQL (presumed to be acceptable). These 5 percent of the lots can be retested before a final decision is made regarding acceptance. The 5 percent retesting rate when the consolidation is presumed to be acceptable is not considered objectionable.

If retesting does incorporate data from the initial series of tests, then for the sample size of 16, the minimum acceptance constant, k, required would be 1.13. Then the following condition would need to be satisfied:

$$Q_L = \frac{\bar{X} - L}{S} \geq k = 1.13$$

where: $\bar{X}$ = mean for sample size of 16

S = standard duration for sample size 16

d. Test Procedure

It is recommended that density measurement be performed according to ASTM Designation: C1040-85 (Standard Test Methods for Density of Unhardened and Hardened Concrete in Place by Nuclear Methods). The ASTM procedure incorporates two test methods: Method A is for direct transmission measurements and Method B is for backscatter measurements. Both methods require automatically timed readings for a period of 1 min. The density is then obtained by using the adjusted calibration curve or by using the unadjusted calibration curve and then applying the calibration adjustment factor.

For each lot being tested, a test should be performed to determine the reference unit weight at 100 percent consolidation. The reference unit weight can be determined using the procedures of ASTM Designation: C 138 (Standard Test Method for Unit Weight, Yield, and Air Content (Gravimetric) of Concrete). The procedure allows use of rodding or internal vibration for consolidating concrete having a slump of 1 to 3 inches (25 to 75 mm). Care should be exercised during consolidation to ensure that concrete is fully (100 percent) consolidated.

The degree of consolidation is then determined simply by dividing the measured density by the reference unit weight.

e. Corrective Measures

When a lot is considered as being not acceptable, then effort should be made to identify the cause of the problem. If the concrete is found to be underconsolidated in localized areas, then hand-held vibrators may be used to adequately consolidate the concrete. If underconsolidation is found to be widespread, then the paving should be stopped until measures are taken to correct the problem resulting in underconsolidation of concrete.

Revibration can be carried out successfully up to about 4 hours from the time of mixing.[56] Actually, revibration at 1 to 2 hours after placement may also result in an increase in the 28-day compressive and flexural strengths.

f. Pavement Type Consideration

No distinction is made in the model acceptance plan for the different pavement types. The requirement of acceptable degree of consolidation is considered equally applicable to jointed plain, jointed reinforced, and continuously reinforced concrete pavements.

5. Adjusted Pay Schedules

Adjusted pay schedules are often used in highway construction to enable acceptance of a product that is only slightly deficient, but at a reduced payment. A construction item that falls just short of the specified quality level does not warrant rejection but neither does it deserve 100 percent payment.[52] There are basically two types of pay schedules, stepped and continuous. Detailed discussions of development of pay schedules are given in references 51, 52, 54, and 57.

Development of a pay schedule requires establishment of the acceptable quality level (AQL) and the rejectable quality level (RQL). For development of an acceptance plan based on degree of concrete consolidation, AQL is set at 10 percent defective and RQL is set at 50 percent defective. Then, payment for material supplied with percent defective less than or equal to AQL is usually at 100 percent of the unit price. Payment for material with percent defective at the RQL is usually between 40 and 60 percent of the unit price. However, any lot with percent defective greater than RQL may be considered unacceptable and may be required to be removed and replaced at the expense of the contractor.

The following discussion is based on use of a continuous pay schedule because it is less ambiguous than a stepped pay schedule. A continuous pay schedule can be defined as follows:

$$PF = P2 + \left(\frac{P1 - P2}{RQL-AQL}\right)(RQL-PD) \qquad (18)$$

where: PF = percent pay factor for percent defective equal to PD.
AQL = acceptable quality level.
RQL = rejectable quality level.
P1 = percent pay factor at AQL.
P2 = percent pay factor at RQL.

For AQL = 10 percent, RQL = 50 percent, P1 = 100 percent, and P2 = 50 percent, equation 18 reduces to

$$PF = 110 - 1.25\ PD \qquad (19)$$

The above pay schedule provides for payment of 110 percent of the unit price when the percent defective is measured to be zero and for payment of 100 percent for percent defective equal to AQL. However, a problem exists with this type of a pay schedule: the contractor who consistently produces at AQL would still have the material rejected 5 percent of the time and thus qualify for full payment only 95 percent of the time. However, for the purpose of the model acceptance plan, equation 18 is considered applicable. The pay schedule determined based on equation 18 is as follows:

Percent Defective	Pay Factor (percent)
0	110.0
10 (AQL)	100.0
20	85.0
30	72.5
40	60.0
>>50 (RQL)	50.0

A pay factor above 100 percent can be used to establish credit to offset pay factors less than 100 percent for lots placed that day. However, under this provision, the total payment for any billing period would not exceed 100 percent.

The percent defective for a given sample size can be estimated using the procedure described in reference 55. The percent defective is associated with the Q_L statistic which is determined according to equation 15 or equation 17. The estimated percent defective for a sample size 8 and corresponding to a given Q_L is given in table 30. The pay schedule based on Q_L is given in table 31. It is seen from table 31 that to qualify for 100 percent payment, the sample mean, $\bar{X}$, for degree of consolidation for the lot should be equal to 98.9 when a lower specification limit for degree of consolidation of 97 percent is used.

6. Application of the Consolidation Monitoring Device (CMD)

The consolidation monitoring device (CMD) is a nuclear backscatter device which continuously and automatically monitors the degree of consolidation of plastic concrete. Details of the CMD are presented in section 4 of this report. Since the CMD is constantly moving over the pavement, it provides an average reading of the density over a large volume of concrete. The CMD currently provides the density information in the form of a strip chart from which densities can be read to the nearest 0.5 pcf (8 kg/m^3).

There are two possible ways to make use of the CMD device in determining the acceptability of concrete consolidation. The CMD can be used primarily to

Table 30. Estimated percent defective for sample size, N=8

Q	0.00	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
0.0	50.00	49.62	49.24	48.86	48.49	48.11	47.73	47.35	46.97	46.59
0.1	46.22	45.84	45.46	45.08	44.71	44.33	43.96	43.58	43.21	42.83
0.2	42.44	42.08	41.71	41.34	40.97	40.59	40.22	39.85	39.48	39.11
0.3	38.75	38.38	38.01	37.65	37.28	36.92	36.55	34.19	35.83	35.47
0.4	35.11	34.75	34.39	34.04	33.68	33.33	32.97	32.62	32.27	31.92
0.5	31.57	31.22	30.87	30.53	30.18	29.84	29.50	29.16	28.82	28.48
0.6	28.15	27.81	27.48	27.15	26.82	26.49	26.16	25.83	25.51	25.19
0.7	24.86	24.54	24.23	23.91	23.59	23.28	22.97	22.66	22.33	22.04
0.8	21.74	21.44	21.14	20.84	20.54	20.24	19.95	19.66	19.37	19.08
0.9	18.79	18.51	18.23	17.95	17.67	17.39	17.12	16.85	16.57	16.31
1.0	16.04	15.78	15.51	15.25	15.00	14.74	14.49	14.24	13.99	13.74
1.1	13.49	13.25	13.01	12.77	12.54	12.30	12.07	11.84	11.61	11.39
1.2	11.17	10.94	10.73	10.51	10.30	10.09	9.88	9.67	9.47	9.26
1.3	9.06	8.87	8.67	8.48	8.29	8.10	7.91	7.73	7.55	7.37
1.4	7.19	7.02	6.85	6.68	6.51	6.35	6.19	6.03	5.87	5.71
1.5	5.56	5.41	5.26	5.12	4.97	4.83	4.69	4.56	4.42	4.29
1.6	4.16	4.03	3.91	3.79	3.67	3.55	3.43	3.32	3.21	3.10
1.7	2.99	2.89	2.79	2.69	2.59	2.49	2.40	2.31	2.22	2.13
1.8	2.04	1.96	1.88	1.80	1.72	1.65	1.58	1.51	1.44	1.37
1.9	1.31	1.24	1.18	1.12	1.07	1.01	0.96	0.91	0.86	0.81
2.0	0.76	0.72	0.67	0.63	0.59	0.55	0.52	0.48	0.45	0.42
2.1	0.39	0.36	0.33	0.30	0.28	0.26	0.23	0.21	0.19	0.17
2.2	0.16	0.14	0.13	0.11	0.10	0.09	0.08	0.07	0.06	0.05
2.3	0.04	0.04	0.03	0.02	0.02	0.02	0.01	0.01	0.01	0.00
2.4	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

Table 31. Pay schedule based on Q_L

Pay Factor, Percent	Required Q_L	Required Sample Mean, $\bar{X}$, Percent
100 (AQL)	1.25	98.9
85	0.85	98.3
72.5	0.54	97.8
60	0.26	97.4
50 (RQL)	0.00	97.0

Notes:
1. Required sample mean, $\bar{X}$, for degree of consolidation is computed using equation 17 with a lower specification limit on degree of consolidation of 97 percent and assuming a sample standard deviation of 1.5 percent.
2. Required Q_L is obtained from table 29.

monitor consolidation or it can be used directly as part of the acceptance sampling plan.

a. CMD as a Monitoring Device

The CMD can be used primarily as a device to monitor concrete consolidation. In this case a benchmark (lower-limit) value of degree of consolidation (density) is established, such as 97 percent degree of consolidation. The benchmark value is identified on the strip chart. When more than 5 percent of the readings fall below the benchmark level, the contractor is notified and requested to make appropriate changes in his paving operation to ensure compliance with the requirements of consolidation.

b. CMD for Acceptance Sampling

The CMD can also be used directly for acceptance sampling of concrete consolidation. However, to make efficient use of the CMD, it would be essential that a digitized form of the density information also be available, possibly on the basis of a time interval, and that a capability exist to automatically provide the mean and the standard deviation for a group of the digitized readings. Then, a procedure for acceptance sampling similar to that described in section 5 can be used. With the CMD's ability to sample at high frequencies, there is greater flexibility in establishing the sample size and/or the lot size and the seller's risk (α) can be greatly reduced while maintaining the same buyer's risk (β).

As an example, assume that the probability of accepting a lot that is 10 percent defective (AQL) is 95 percent. Then, the minimum quality index, Q_L, required and the achievable RQL at probability of acceptance of about 5 percent for different sample sizes are as follows:

Sample size, n	Minimum Q_L	RQL, percent	P_a (RQL)
20	0.88	35	3
30	0.92	30	4
50	0.99	25	3
100	1.08	20	3

P_a(RQL) = probability of acceptance when percent defective is at RQL = β

If a lower limit for acceptance of degree of consolidation of 97 percent is established and a sample standard deviation of 1.5 percent is assumed for all the sample sizes, the following sample mean would be required for a lot to be considered acceptable:

Sample Size	Minimum Q_L	Sample Mean, $\bar{X}$, percent
20	0.88	98.3
30	0.92	98.4
50	0.99	98.5
100	1.08	98.6

An adjusted pay schedule can also be developed for the different sample sizes using the procedures discussed in section 5.

A great advantage of the CMD is that lot size can be as small as can be handled administratively and full use would be made of all the digitized density data that is obtained. The rate for digitizing can be related to the lot size, so that the number of measurements digitized per lot equal the selected sample size.

FIELD TRIALS OF CONSOLIDATION ACCEPTANCE PLAN

The FHWA contract called for field trials of the model acceptance plan for consolidation at two sites. At one site, use of a conventional nuclear gauge was proposed, at the other site, use of the CMD. During August 1986, the first field trial, using a conventional nuclear gauge, was carried out at a site on Interstate 86 in Idaho. When a suitable field test could not be scheduled for the CMD during late 1986/early 1987, its trial was replaced with a second conventional gauge test. The second field trial was carried out in Iowa in May 1987.

1. Field Trial in Idaho

An 11-mile (19 km) section of Interstate 86 in Idaho was constructed during the summer of 1986 between Raft River and the Rockland junction west of American Falls. The pavement design prepared by the Idaho DOT required a 10 inch (250 mm) thick plain concrete pavement to be placed over an existing asphalt treated base. The pavement was 38 ft (11.6 m) wide and incorporated an 8.5-ft (2.6-m) wide tied outside concrete shoulder and a 3.5-ft (1.1-m) wide inside concrete shoulder. Joints were spaced at 13, 15, 16, and 14 ft (4.0, 4.6, 4.9, and 4.3 m). A Guntert and Zimmerman slipform paver was used to place the 38 ft (11.6 m) width of concrete. Spud vibrators on the paver were spaced at 18-inch (460 mm). An automatic dowel bar inserter supplied by Gomaco was used to insert 1-1/4 inch (33 mm) diameter by 18-inch (460 mm) long dowel bars at 12 inch (305 mm) intervals transversely. The inserter unit followed the slipform paver.

a. Field Trial Details

Field trial of the model acceptance plan was carried out on August 27 and 28, 1986, along the westbound lanes of I-86. The following items of work were accomplished:

1. Eight lots were tested for consolidation.
2. The nuclear density gauge was calibrated for the concrete used at the project.
3. A study was conducted to establish correlation between direct transmission and backscatter measurements.
4. The practicality of consolidation monitoring was evaluated.

A nuclear density gauge (Model MC-1) manufactured by Campbell Pacific Nuclear Corporation, Pacheco, California, was used. An Idaho DOT technician operated the gauge, generally following the procedures of ASTM Designation: C 1040 (Density of Unhardened and Hardened Concrete in Place by Nuclear Methods).

The pavement to be tested was divided into lots. Eight readings, as required by the proposed plan, were obtained per lot. Test locations were randomly selected across the width of the pavement and were spaced at about 42 ft (13 m) longitudinally. A platform that was a part of the Gomaco dowel inserter unit was used to facilitate testing. The inserter unit stopped at every joint location and remained stationary for about 12 to 13 seconds while the dowels were being inserted into the plastic concrete. A density test was conducted at every third stop of the inserter unit, about every 42 ft (13 m). A test is shown in progress in figure 27. For each test, the exact location across the width of the pavement was selected using a random number table.

In order not to disturb the paving operation, direct transmission readings were obtained using a 15-second count period. An 8-inch (200 mm) probe depth was used. The nuclear gauge was calibrated for the I-86 concrete using an 18-inch (460 mm) square and 10-inch (250 mm) deep box. The small size box was used because of the limitation of the weight scale in the field (maximum of 350 lb (160 kg)). The calibration test indicated that a correction factor of 1.026 should be applied to the manufacturer supplied calibration tables.

b. Test Results

Results of the consolidation monitoring for the eight lots tested from the dowel inserter are given in table 32. Table 32 shows that the concrete for the eight lots tested was well-consolidated and met the acceptance criteria for the value of quality index, Q_L of 0.66.

A second study was then conducted to determine the correlation between direct transmission and backscatter measurements. Test sites were located near the pavement edge, thus allowing the technician to place the gauge while standing on the unpaved shoulder. The following measurements were made at each test location:

1. Direct Transmission - 8 inch (200 mm) depth probe.
   - 15 second count.

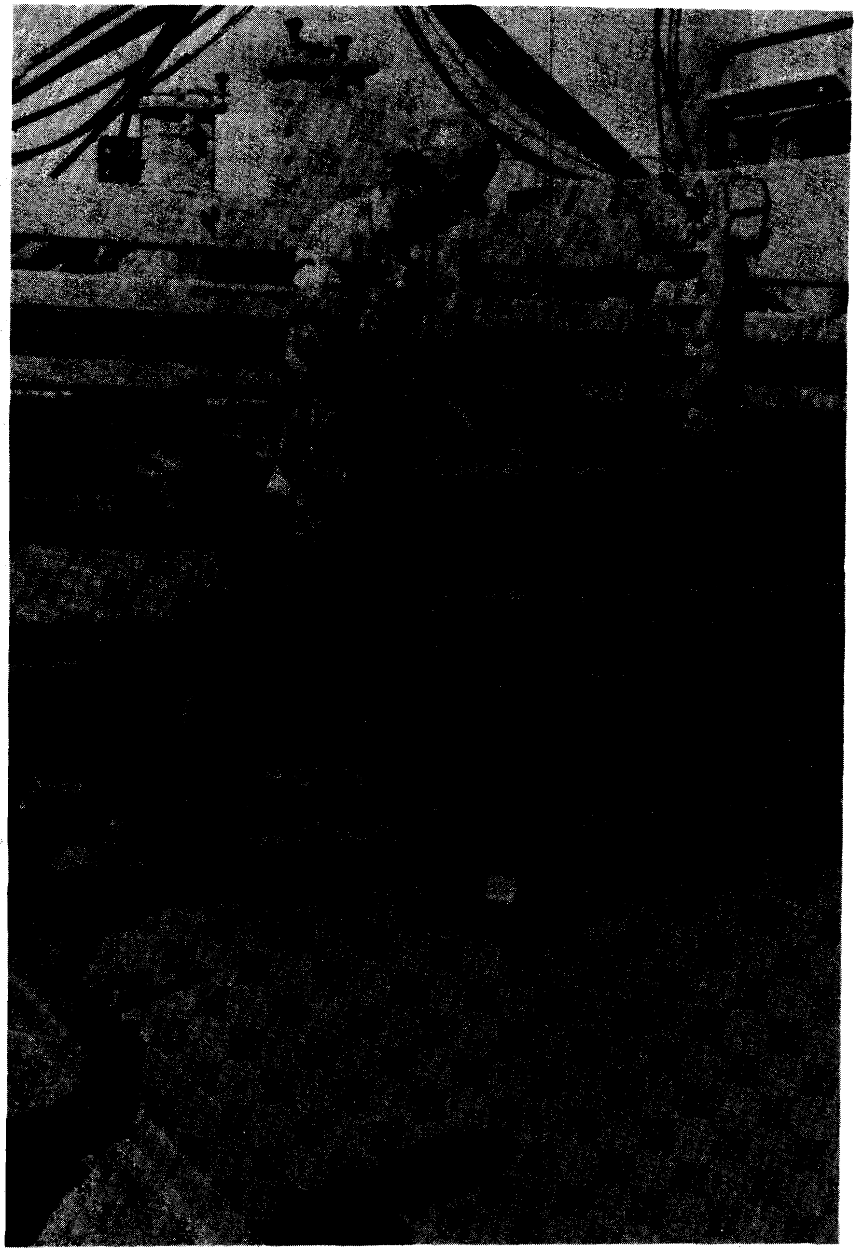


Figure 27. Consolidation testing in progress - Idaho site.

Table 32. Results of field trial - Idaho site.

Lots	No. of Tests	Average Measured Density, pcf[1]	Reference Density[2], pcf	Average Consolidation, %	Standard Deviation %	Q_L[3]
A	8	142.3	143.9	98.9	0.5	3.8
B	8	142.0	143.9	98.7	1.3	1.3
C	8	143.5	143.9	99.7	0.8	3.4
D	8	143.0	143.9	99.4	1.1	2.2
E	8	144.1	143.9	100.2	1.3	2.5
F	8	143.5	143.6	99.9	0.9	3.2
G	8	143.0	143.6	99.6	0.8	3.3
H	8	143.6	143.6	100.0	0.9	3.3

Notes: 1. $kg/m^3 = lb/ft^3 \times 16.018$

2. Reference density is average of rodded unit weight measured at the time of concrete quality assurance testing.

3. $Q_L = \frac{\bar{X} - 97}{S} \geq 0.66$ (for acceptance)
(Lower limit for acceptance of consolidation is 97%)

2. Direct transmission - 8 inch (200 mm) probe depth.
- 1 minute count.

3. Backscatter - 15 second count.

A total of thirty (30) sets of measurements were made along the inside edge of the pavement. Test results are given in table 33. A reasonably good correlation is obtained between the two types of the direct transmission measurements. As expected, the backscatter measurements were generally higher than the direct transmission measurements.

2. Field Trial in Iowa

A 9-mile (15.8 km) section of U.S. 520 in Iowa was constructed during 1987 east of Coalville near Fort Dodge. The pavement design prepared by the Iowa DOT required a 9-inch (230 mm) thick plain concrete pavement to be placed over a 4-inch (100 m) thick slipformed lean concrete base. The concrete pavement was 24 ft (7.3 m) wide and the lean concrete base was 28 ft (8.5 m) wide. Joints were uniformly spaced at 20 ft (6.1 m). Basket assemblies were used to place 1-1/4 inch (32 mm) diameter and 18-inch (460 mm) long dowel bars. Concrete was first spread onto the base using a side-dump spreader unit. A Caterpillar SF-550 slipform paver then provided the finished surface. On this project supplementary vibration was provided at dowel basket assemblies. This was done by a worker using a hand-held spud vibrator and positioned on a platform on the spreader unit.

a. Field Trial Details

Field trial of the model acceptance plan was carried out on May 7, 1987, along the eastbound lanes of U.S. 520. Paving was progressing west to east. To enable nuclear gauge testing, the contractor had provided an independent platform (bridge) unit. The platform unit was 2 ft (0.6 m) wide and about 18 inches (460 mm) high over the finished pavement surface. The platform was not powered and had to be pulled or pushed. The platform had been placed behind the tining/curing bridge, although ideally it would have been placed between the paver and the tining/curing bridge. Because of the platform's position, it had to be moved back frequently to allow curing compound application. The whole process proved to be too time consuming so a decision was made to only use the platform to facilitate Iowa DOT's consolidation testing over dowel basket assemblies. As a result, a platform on the paver was used for consolidation testing related to the acceptance plan. A 15-second

Table 33. Results of the correlation study - Idaho site.

Test No.	Measured Density, pcf		
	Direct Transmission 15 sec. count	Direct Transmission 1 min. count	Backscatter 15 sec. count
1	139.5	139.0	143.1
2	139.5	140.4	146.5
3	142.1	141.6	145.7
4	141.6	141.6	144.7
5	140.9	142.4	142.4
6	145.7	145.2	144.2
7	140.4	139.5	142.6
8	140.0	139.5	143.1
9	141.1	142.9	144.2
10	143.1	143.1	142.1
11	145.2	144.7	147.7
12	144.7	144.7	147.7
13	143.6	143.6	142.6
14	146.5	146.2	144.5
15	143.6	143.1	145.2
16	142.6	142.6	140.9
17	141.9	142.1	145.2
18	143.4	143.6	146.7
19	140.9	142.1	144.7
20	143.4	143.1	147.2
21	143.6	142.1	146.5
22	142.6	140.9	143.1
23	143.1	142.1	148.8
24	140.4	141.4	146.7
25	142.6	141.4	145.7
26	141.1	141.9	144.5
27	140.6	139.0	143.1
28	140.4	140.6	140.8
29	141.6	141.6	143.6
30	142.4	142.6	147.7
Average	142.3	142.2	144.7
St. Dev.	1.8	1.8	2.1

$kg/m^3 = lb/ft^3 \times 16.018$

count was used for each measurement. The gauge was placed in front of the platform and was picked up behind the platform as the paver moved forward. There was no disruption to the paving operation. A view of the consolidation testing using the independent platform is shown in figure 28.

A nuclear density gauge (Model 3401-B) manufactured by Troxler Electronic Laboratories, Inc., Research Triangle Park, North Carolina, was used. The gauge had been previously calibrated for concrete testing by Iowa DOT. The procedures of ASTM Designation: C 1040 were generally followed.

Because of time constraints, only four lots were tested. Thus, a total of 32 measurements were made.

Test locations were spaced at about 20-ft (6 m) intervals. Thus, a lot size equalled about 160 ft (49 m). Test locations were randomly selected across the width of the pavement. An 8-inch (200 mm) probe depth was used.

b. Test Results

Results of the consolidation monitoring for the four lots tested are given in table 34. The table shows that the as-placed concrete was well consolidated and met the acceptance criteria for the value of quality index, Q_L, of 0.66.

3. Discussion of Field Trials

Some of the highlights of the field trials are summarized below:

1. Field monitoring of concrete consolidation is considered practical, and, if properly planned, will not interfere with the contractor's paving operation.
2. Ideally, a separate platform (bridge) should be used to facilitate the consolidation testing. The deck of the platform should be low enough to facilitate repeated placement of the "heavy" nuclear gauge on the pavement surface. On large paving projects, such a platform is maintained by the contractor.
3. Pavement to be tested should be plain concrete with a low-slump.
4. Lot size can be adjusted in the field to correspond with the speed of paving.
5. Frequent tests should be conducted to obtain the reference unit weight of the plastic concrete. In addition, a carefully controlled calibration check should be made for each nuclear gauge using the

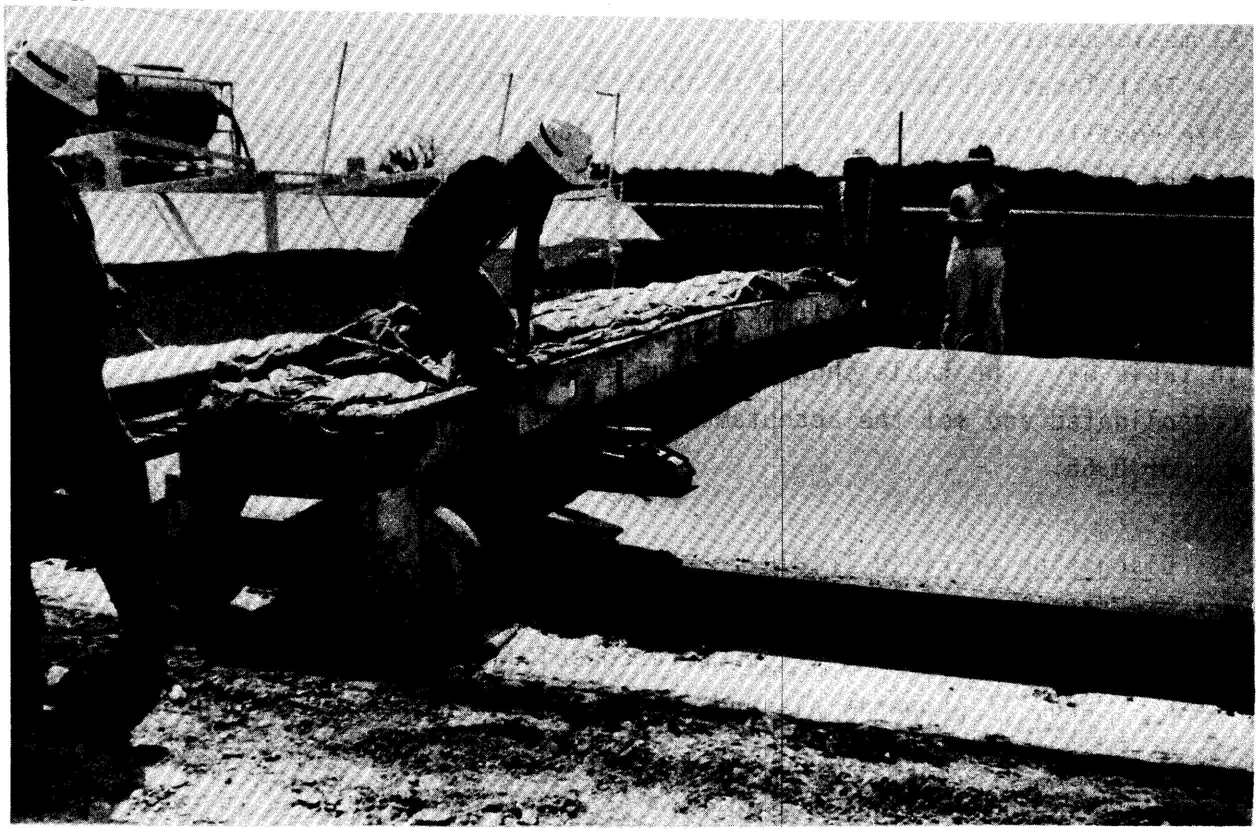


Figure 28. Consolidation testing in progress - Iowa site.

Table 34. Results of field trial - Iowa site.

Lots	No. of Tests	Average Measured Density, pcf[1]	Reference Density[2], pcf	Average Consolidation, %	Standard Deviation %	Q_L[3]
A	8	144.3	143.9	100.3	1.3	2.5
B	8	143.0	143.9	99.4	1.4	1.7
C	8	144.0	143.9	100.0	1.6	1.9
D	8	144.0	143.9	100.0	1.6	1.9

Notes: 1. $kg/m^3 = lb/ft^3 \times 16.018$

2. Reference density is average of rodded unit weight measured at the time of concrete quality assurance testing.

3. $Q_L = \frac{\bar{X} - 97}{S} \geq 0.66$ (for acceptance)
(Lower limit for acceptance of consolidation is 97%)

project concrete at the start of the paving project following the procedures of ASTM Designation: C 1040. The weight scales used in the field should be frequently calibrated.

4. Cost/Benefit Considerations

As demonstrated in the Idaho and Iowa field trials, the cost of consolidation monitoring is not high. On large paving projects, a contractor can provide a separate platform (bridge) to facilitate consolidation testing. As discussed in the next section, the consolidation testing can be carried out intermittently--for one or two days at the start of the project and periodically afterwards and then whenever changes are made in the paving operation or equipment. Thus, the cost of providing the platform would be minimal.

A major cost of consolidation testing is the labor cost which is estimated at $300 per technician day. A licensed technician is required to perform the testing. With advance planning, a qualified technician can be made available at a project site at the start of the paving operation and as needed later on. The acceptance plan developed is simple to use and to interpret.

Considering the minimal cost for conducting consolidation testing, the risk of not obtaining adequate consolidation is significant. As discussed earlier, poor consolidation can result in lower strength concrete and, therefore, reduced service life.

The fact that consolidation test results are immediately available should be emphasized and, therefore, any problems with concrete quality can be quickly corrected with minimal losses in time and money. This is in contrast to strength tests on cores from hardened concrete. Most agencies do not take cores routinely. If cores belatedly confirm concrete is of poor quality, e.g., poorly consolidated, then the concrete may need to be removed or other expensive corrective measures may need to be applied.

Consolidation testing also allows higher sampling frequencies than are available with other diagnostic testing. A nondestructive consolidation test can easily be made at intervals of 40 to 50 ft (10 to 12 m). Obtaining cores from hardened concrete to determine concrete quality would not be nearly as cost-effective.

5. Recommendations for Field Monitoring of Consolidation

The field trials of the proposed acceptance plan for consolidation indicate that it is practical and economically feasible to monitor consolidation of concrete pavements. However, because such monitoring would be a new feature with regards to control of paving concrete quality, it is recommended that highway agencies use the proposed acceptance plan for the first year primarily to monitor concrete consolidation and not for acceptance purposes. Both highway agency and contractor personnel need to become familiar with the necessity and the procedures for controlling concrete consolidation. The one-year trial period would also allow development of a data base on field consolidation and to make revisions, if necessary, in the proposed model acceptance plan.

During the trial period, consolidation can be monitored during the one or two days at the start of the paving operation and then occasionally thereafter. Continuous monitoring is not recommended as being cost-effective at this time. However, additional monitoring could be performed on larger projects when the paver is replaced or extensively repaired or if major changes are made in the paving procedure.

SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

1. Summary and Conclusions

A concrete pavement is a manufactured item. Therefore, it is necessary to ensure that quality control is exercised at all critical phases of production. Presently, quality control specifications exist to assure a quality product through all phases of construction up to delivery of concrete at a site. However, there are no direct specifications available to monitor concrete quality after the concrete is placed in front of the paver.

In this study the importance of achieving proper consolidation of concrete has been demonstrated. Failure to achieve full consolidation can result in significant loss in strength, bond to reinforcing steel, and large increases in permeability of concrete. Compressive strength may be reduced as much as 30 percent for a 5 percent reduction in degree of consolidation. Bond to reinforcing steel is even more dramatically reduced due to inadequate consolidation.

Nuclear density gauges offer a rapid means for controlling densities in the field. Many of these devices are commercially available, and can be used to determine density of surface layers as well as to obtain information on density through the full thickness of the slab. At present, 25 States are using, or have used, nuclear density gauges for monitoring concrete density. Only half of this number, however, use the gauge on a routine basis, mostly applied to density measurements on low slump dense concrete (LSDC) bridge-deck overlays. Use on full depth pavements is minimal at this time.

A consolidation monitoring device (CMD) has been developed which can be mounted on a paver and used to obtain continuous data on consolidation of concrete in the upper 4 inches (100 mm) of the pavement surface. This device has been used on an experimental basis with good results.

Using careful procedures, precision of nuclear density gauge readings can be better than 1 lb/ft^3 (16 kg/m^3). Field data indicate overall variabilities of readings on well-controlled projects to be in the range of 1 to 2 lb/ft^3 (16 to 32 kg/m^3). Improved methods of field calibration are needed in order to realize the full potential of nuclear density devices.

A model acceptance plan for inspection of concrete consolidation has been proposed based primarily on the effect of concrete consolidation on projected pavement service life. The model acceptance plan is based on the inspection by variables form, rather than inspection by attributes. The proposed plan based on buyer's and seller's risks of 5 percent requires a sample size of 8 and a minimum quality level, Q_L, of 0.66. Procedures for acceptance sampling when the CMD is used are also discussed.

Field trials of the proposed acceptance plan for consolidation in Idaho and Iowa indicate that it is practical and economically feasible to monitor consolidation of concrete pavements. Consolidation testing results are immediately available and therefore problems with concrete quality can be quickly corrected without significant losses in time and money.

Study results demonstrated that concrete consolidation is an important property of concrete and is critical to satisfactory performance of concrete pavements. Use of nuclear density gauges or the CMD for inspection of density should be considered to be a practical necessity.

2. Recommendations for Future Research

Analysis of the results of this study has demonstrated the need for further research in this area. The following topics should be considered:

- It is known that, for well-consolidated concrete, such properties as flexural and tensile strength and elastic modulus can be related to compressive strength. In this program, no direct measurements of the effects of consolidation on any mechanical properties other than compressive strength and bond of concrete to reinforcing steel were made. More information is needed regarding the effects of consolidation on other mechanical properties, from which current relationships between compressive strength and these properties could be verified, or modified as needed.

- The stiff, paving grade concrete used in this study made it difficult to study the effects of overvibration and segregation on important properties of concrete. Much work needs to be done in this area using more fluid mixtures which would be more susceptible to overconsolidation.

- Results of this program indicated that degree of consolidation has little impact on resistance of properly air-entrained concretes to freezing and thawing. This may not be true of concretes where lower levels of air content are obtained, or where marginal aggregate sources are used. Additional work in this area would be beneficial.

- Procedures used to establish field reference density standards for calibration of nuclear density gauges vary widely between State highway agencies. A consistent and reliable procedure needs to be developed. In order to function adequately as a standard the reproducibility of such a procedure must be superior to the inherent precision of nuclear density gauges.

- An investigation should be made into recently developed techniques for using scattered radiation to measure density of localized internal volumes. A continuous monitoring device which would be capable of localizing density with depth is feasible.

- Effort should be made to obtain digitized inplace density information for the CMD. The availability of the digitized information together with automatic computation of the mean and standard deviation for a group of readings would make the use of the CMD very attractive for acceptance sampling of concrete consolidation.

- The model acceptance sampling plan presented in this report should be field tested at several projects in different States. Field testing of the plan will provide assessment of the plan in terms of the following:

  - Practicality of the plan.
  - Cost of testing.
  - Identification of potential ambiguities.
  - Effectiveness of the recommended lower limit of 97 percent for the degree of consolidation.
  - Highway agency and contractor responses.

For the first year, the plan should be used primarily to monitor consolidation and to familiarize State highway agency and contractor personnel with the acceptance plan requirements.

APPENDIX A:

Batch Analyses of Concrete Mixtures

Concretes were cast in three rounds prepared on three separate days for each of the six mixtures. Two separate batches of concrete were cast for each round. Table 35 shows calculated batch quantities, using measured air content to determine batch volume.

Table 35. Concrete mix proportions and characteristics

Mix No.	Round	Batch	Aggregate	Proportions: Quantities, lb/yd^3 1/ Cement	Sand (SSD)	Coarse Aggregate (SSD)	Water	Characteristics: W/C Ratio	Slump 2/ (inches)	Air Content %
1	1	A	Limestone	515	1301	1790	247	0.48	2.0	7.1
	1	B	"	523	1320	1816	248	0.47	1.3	5.8
	2	A	"	510	1300	1789	244	0.48	2.0	7.5
	2	B	"	520	1324	1822	237	0.46	2.0	6.3
	3	A	"	521	1329	1828	249	0.46	1.2	5.2
	3	B	"	521	1329	1828	243	0.47	1.9	5.6
2	1	A	"	616	1292	1811	258	0.42	0.9	5.3
	1	B	"	617	1293	1813	258	0.42	1.2	5.2
	2	A	"	618	1295	1816	259	0.42	0.9	5.0
	2	B	"	618	1295	1816	259	0.42	0.9	5.0
	3	A	"	611	1281	1795	256	0.42	0.9	6.2
	3	B	"	610	1278	1792	256	0.42	1.3	6.4
3	1	A	"	609	1239	1737	256	0.42	1.0	8.8
	1	B	"	607	1237	1734	255	0.42	1.4	9.0
	2	A	"	607	1237	1734	255	0.42	1.4	9.0
	2	B	"	618	1259	1764	256	0.42	0.9	7.1
	3	A	"	621	1264	1714	261	0.42	1.4	8.1
	3	B	"	622	1267	1717	261	0.42	1.3	7.9
4	1	A	Gravel	520	1233	1966	218	0.42	1.3	6.1
	1	B	"	516	1222	1949	216	0.42	2.1	7.0
	2	A	"	521	1227	1956	219	0.42	2.0	6.4
	2	B	"	523	1231	1963	214	0.41	2.1	6.4
	3	A	"	519	1220	1946	219	0.42	2.1	6.9
	3	B	"	526	1238	1974	222	0.42	1.3	5.4
5	1	A	"	607	1188	1895	238	0.39	1.9	5.8
	1	B	"	608	1191	1900	242	0.40	1.8	5.3
	2	A	"	614	1198	1911	238	0.39	1.3	5.0
	2	B	"	613	1195	1907	238	0.39	1.3	5.2
	3	A	"	612	1193	1903	238	0.39	2.1	5.4
	3	B	"	611	1191	1899	237	0.39	2.1	5.6
5	1	A	"	609	1162	1853	236	0.39	2.0	7.6
	1	B	"	613	1169	1865	233	0.38	2.0	7.2
	2	A	"	612	1167	1861	234	0.38	1.2	7.3
	2	B	"	612	1167	1861	234	0.38	1.5	7.3
	3	A	"	612	1166	1860	235	0.39	1.2	7.3
	3	B	"	612	1166	1860	235	0.39	1.0	7.3

1/ $kg/m^3 = lb/yd^3 \times 0.5933$
2/ mm = in. x 25.4

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